TECHNICAL REPORT C-69-4

USE OF EPOXY OR POLYESTER RESIN CONCRETE IN TENSILE ZONE OF COMPOSITE CONCRETE BEAMS

by

H. G. Geymayer

March 1969

Sponsored by

Assistant Secretary of the Army (R&D)
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Conducted by

U. S. Army Engineer Waterways Experiment Station
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FOREWORD

A research investigation, "Feasibility Study on Epoxy and Polyester Resin and Portland-Cement Concrete Beams," sponsored by the Assistant Secretary of the Army (R&D), was authorized by a memorandum to the Chief, Concrete Division, U. S. Army Engineer Waterways Experiment Station (WES), dated 10 November 1965, File WESVB, Subject: "In-House Laboratory Initiated Research Program, FY 66."

The work was performed at the WES Concrete Division during the period January 1966 to December 1967, under the direction of Messrs. Bryant Mather, James M. Polatty, Dr. Helmut G. Geymayer, SP 5 William E. Walker, and SP 4 William D. Smart. This report was prepared by Dr. Geymayer.

Directors of the WES during the investigation and the preparation of this report were COL John R. Oswalt, Jr., CE, and COL Levi A. Brown, CE. Mr. J. B. Tiffany was Technical Director.

CONTENTS

CONVERSION FACTORS, BRITISH TO METRIC UNITS OF MEASUREMENT

British units of measurement used in this report can be converted to metric units as follows:

Multiply	By	To Obtain
inches	2.54	centimeters
feet	0.3048	meters
square inches	6.4516	square centimeters
cubic feet	0.0283168	cubic meters
cubic yards	0.764555	cubic meters
quarts	0.946353	cubic decimeters
gallons (U.S.)	3.785412	cubic decimeters
pounds	0.45359237	kilograms
tons (2000 pounds)	907.185	kilograms
pounds per square inch	0.070307	kilograms per square centimeter
pounds per cubic foot	16.0185	kilograms per cubic meter
foot-pounds	0.138255	meter-kilograms
pounds per cubic inch	27679.91	kilograms per cubic meter
Fahrenheit degrees	5/9	Celsius or Kelvin degrees*

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

SUMMARY

This report describes the results of an investigation into the feasibility of combining the high compressive strength of portland cement concrete and the superior tensile strength of epoxy or polyester resin concrete into a composite beam. This would increase the beam's flexural strength and improve the corrosion protection for the reinforcement at large deflections by eliminating tensile cracks.

The report describes in detail the development of high-strength resin concrete mixtures and summarizes the most important engineering properties of the selected mixtures. Also included are the results of third-point loading tests of 12 reinforced and unreinforced composite beams with 1-1/2- and 3-in.-thick layers of epoxy and polyester resin concretes. These results are compared with results of tests of two reference beams without resin concrete layers and with analytical results.

The study led to the following principal conclusions:

- a. Properly designed resin concrete layers at the tension face of concrete beams can be used to moderately increase the strength and rigidity of reinforced concrete beams, or to upgrade the flexural strength of unreinforced beams by a factor of two to three.
- b. More important than their influence on strength is the ability of resin concrete layers to provide a noncracking moisture barrier or corrosion protection practically up to beam failure.
- c. The epoxy resins appeared to be more suitable for this application than the polyester resins investigated due to lower shrinkage and exotherm as well as higher tensile strength and tensile strain capacity.
- d. In proportioning resin concrete mixtures, early attention should be directed to properties other than strength (such as shrinkage, exotherms, coefficient of thermal expansion, creep, sensitivity to environmental factors, etc.).

USE OF EPOXY OR POLYESTER RESIN CONCRETE IN TENSILE ZONE OF COMPOSITE CONCRETE BEAMS

PART I: INTRODUCTION

Background

1. During the past decade or two, resin concretes and resin mortars have generated ever-increasing interest throughout the building community; and as a result, considerable data on different resin concrete* mixtures have been forthcoming from research laboratories all over the world. Although a large variety of resin-hardener systems has been investigated for this purpose, the vast majority of the work has been principally concentrated on three groups of resins:

- a. Epoxy resins.
- b. Polyester resins.
- c. Furanic resins.

This country has taken the lead in the development and study of the first two resin groups for civil engineering applications, while some European laboratories have concentrated on the lower strength, but more economical, furanic resins, apparently with fair success.[1-5]

2. From the growing accumulation of individual data, a technology is now beginning to evolve, comparable to the well-established conventional concrete or asphalt technology; in fact, certain basic relations have already been established. However, it appears that most studies to date have been restricted essentially to what one might call "basic mixture proportioning." For this reason, relatively little is known about the performance of particular resin concretes or mortars in the environments of their potential use, especially over any length of time. Several studies have been undertaken on the influence of resin modifier and hardener type and content, as well as aggregate mineralogy, shape, moisture content, and

* The term resin concrete is applied to concretes using resins in lieu of portland cement as a binder for the aggregate particles.

grading, upon the static compressive strength of standard cubes and cylinders, the elastic modulus (E), the rupture modulus, etc. Not until fairly recently, however, was it realized that resin concretes can be extremely sensitive to placing, curing, and testing conditions[5,6] (e.g. relative humidity, temperature, loading rate, and specimen shape and size), obviously much more so than conventional concretes. This sensitivity makes a comparison and evaluation of test results from different laboratories difficult. Also, little is known about the important long-term behavior of resin mortars in different environmental situations; thus, applications of the new construction material in practice have been slow and very limited in scope. This is, of course, largely a result of the still comparatively high costs of resins, especially epoxies. The majority of practical applications of resin mortars to date has been in the field of grouting[7,8] and concrete repair,[9-13] especially repair of concrete pavement and bridges.[4,5,14,15] True structural applications, i.e. important load-carrying uses, of resin concretes or mortars have been scarce and cautious and involved only small volumes of material. The few actual structural applications known to the author were made in the construction of composite steel-concrete bridges where pure epoxy resins and resin mortars have been used to bond concrete decks to steel girders;[16,17] also, these resins have been used in prefabricated concrete construction to join individual parts to ensure their monolithic action.[18-23] The designers, however, were usually careful not to rely entirely on the resin mortar for the safety of their structures and, more often than not, provided steel connectors for good measure.[16,18]

3. One can safely say that the potential of resin concrete as a structural material is just beginning to be explored. Considering the high tensile strength (f_u) to compressive strength ratio, the excellent corrosion resistance, and the possible low permeability of these concretes (in addition to high compressive strength, good bonding characteristics, and rapid setting time) a wide field of structural applications can easily be visualized--if it were not for the punishingly high costs. It should, perhaps, be remembered that a large number of structures, in addition to carrying external loads, are exposed to rather severe physicochemical

environments that necessitate very expensive auxiliary measures to protect conventional construction materials (such as portland cement concrete, masonry, or steel) from premature degradation. Provisions to protect the load-bearing structure from an aggressive environment are sometimes more expensive than the structure itself. Therefore, it seems logical to search for a material that combines the function of carrying the load and protecting the structure from the particular aggressive environment. Resin concrete appears very capable of fulfilling this double function in the majority of cases. In fact, it can well exceed conventional structural materials in strength and at the same time surpass standard protective materials in corrosion resistance and impermeability. In addition, resin concrete develops its full strength in a very short time, just the opposite of conventional concrete.

4. From the above, it follows that resin concrete should not be regarded as a potential substitute for all conventional concrete since resins probably will always be more expensive than portland cement. But in cases where conventional concrete is incapable of giving the desired combination of strength and corrosion or moisture protection, resin concretes could play an increasingly important role.

5. The study reported herein is an attempt to combine conventional concrete and resin concretes in a composite structural member so that the advantages of both materials can be utilized to achieve an optimum solution from a technical and economic standpoint. The idea is to replace part of the conventional concrete in the tensile zone of a beam with resin concrete, thereby taking full advantage of the higher f_u of this material without increasing the overall cost to an unacceptable level. A resin concrete layer at the bottom of a beam in which the tensile reinforcement is embedded should help in bond and shear and also eliminate cracking normally inherent in heavily loaded, reinforced concrete beams, thus significantly reducing the threat of corrosion of the reinforcing steel, particularly in highly aggressive environments (e.g. desalination plants, maritime construction, etc.). On the underside of a slab, such a layer could serve as an integral surface protective layer, shielding the structure from moisture and chemical attack while contributing to its strength.

Due to the considerably higher strength of resin concretes as compared to conventional concrete (though usually at a relatively low E and high creep), a significant increase in the load-carrying capacity could be hoped for, possibly even allowing a reduction in the physical size or the amount of reinforcement in the member. Reinforcing rods with poor bonding characteristics (plain steel bars, fiberglass rods, etc.) could probably be used also.

6. The concept of a reinforced, composite portland cement and resin concrete member, if proven feasible and worthwhile, could certainly be extended to other configurations (e.g. sandwich construction), different applications (e.g., repair and strengthening of structures), and structural elements. However, it is the flexural member that should, in theory, exhibit the most beneficial effect. As a result, this feasibility study was limited to simply supported beams with a rectangular cross section and two fairly typical resin systems.

Objective and Scope

7. The objective of this pilot program was to investigate the feasibility of using a layer of high-strength, corrosion-resistant, impermeable resin concrete in the tensile zone of reinforced concrete beams in order to improve their strength (or make a reduction of reinforcement possible), increase their resilience, and increase their resistance to aggressive environments as well as to facilitate the use of reinforcing materials with poor bond characteristics and/or chemical compositions incompatible with portland cement concrete. A secondary objective was to develop resin concrete mixtures and procedures suitable for such applications.

8. The scope of the investigation was restricted to two particular resins, i.e. a two-component polysulfide-epoxy compound and a polyester resin-methyl ethyl ketone peroxide catalyst system.

PART II: TESTING PROGRAM

9. The experimental program was conducted in the following three phases.

a. The first phase consisted of the design of resin concretes and the evaluation of their physical properties. With the two resin-hardener systems chosen as binder materials, a 3/8-in.* maximum size, rounded quartz-chert aggregate was selected for the main program. Several test series were performed to determine an optimum aggregate gradation and resin content for the two resin concretes. Eventually the values of some of the most important physical properties of both optimum mixtures (such as compressive strength, flexural strength, tensile strength, stress-strain characteristics, shrinkage behavior, etc.) were established. Additional tests were undertaken with crushed limestone aggregate. Excellent strength results for a polyester concrete with limestone aggregate were obtained on standard laboratory specimens; however, due to its excessive shrinkage, the polyester concrete with limestone aggregate could not be used successfully in the main beam program. (See paragraphs 37 and 62).

b. Reinforced and unreinforced concrete beams with epoxy concrete layers were fabricated and tested. Nine 78- by 9- by 4-in. simply supported beams with different reinforcement and 1-1/2- and 3-in.-thick layers of epoxy concrete (table 1) were made and tested under third-point loading. The results of these tests were then compared with the results obtained from a conventional reference beam without an epoxy concrete layer.

c. In the last phase, reinforced and unreinforced concrete beams

* A table of factors for converting British units of measurement to metric units is presented on page ix.

were fabricated with polyester concrete layers and tested. Part of the previous test series, involving a total of four beams with resin concrete layers and a reference beam, was repeated using the more economical polyester resin concrete, and the results were compared with those of the reference beam (table 1).

Materials and Techniques

Epoxy and polyester resins

10. The following two resin-hardener systems were used for this study.

- a. A two-component polysulfide-epoxy compound (1:1 by volume) having an amber color and a syruplike consistency (price: about $13 per gallon or $1.15 per lb).
- b. A two-part polyester resin-methyl ethyl ketone peroxide catalyst diluted with 60% dimethyl phthalate and having an almost waterlike appearance and viscosity (price: about $0.38 per lb for small quantities).

Aggregate

11. Dry, clean quartz-chert aggregate was used for all but a few resin concrete mixtures to ensure good bonding characteristics with the resin matrix[3] and minimum shrinkage. In order to obtain a minimum void content between the aggregate particles and thus achieve the most economical resin concrete with a relatively high elastic modulus and good strength, two test series were performed to select a maximum bulk density grading for both a continuously graded 3/8-in. maximum size aggregate and a gap-graded aggregate with the same maximum size.

12. Continuous grading. The aggregate was graded [illegible] seven fractions (3/8-in. to No. 4, No. 4 to No. 8, No. 8 to No. 16 [illegible] 16 to No. 30, No. 30 to No. 50, No. 50 to No. 100, and passing the No. 100 sieve) and an exponential sieve curve

$$A = (d/D)^n \quad \text{(reference 24)}$$

where

A = amount of material passing sieve opening d (in %)

D = maximum size aggregate

d = variable sieve opening

Four different values (between 0.2 and 0.5) for the exponent n were tried, and the bulk density of the resulting aggregate mixtures was determined. It was found that the greatest aggregate compaction, i.e. minimum void content, could be obtained with an exponent of about 0.4, i.e. slightly below the exponent that yields the familiar Fuller parabola (n = 0.5) (see plate 1). This result agrees well with earlier findings for alluvial sand-gravel mixtures.[24] The grading used for all continuously graded resin concretes is tabulated below.

Passing Sieve	Retained on Sieve	Percentage by Weight
3/8-in.	No. 4	25.2
No. 4	No. 8	17.3
No. 8	No. 16	13.9
No. 16	No. 30	10.6
No. 30	No. 50	8.0
No. 50	No. 100	6.5
No. 100		18.5

13. Gap grading. According to the theory of packed spheres, the diameter, d , of a small sphere that will slip through the gaps between densely packed larger spheres of a constant diameter, D , (in octahedral or tetrahedral configuration) cannot exceed

$$d = 0.155D \quad \text{(reference 24)}$$

In practice, however, it is recommended[24] to reduce d to at least 0.14D since aggregates are not truly spherical and will be surrounded by a binder matrix that further decreases the size of the gaps. Thus, taking the minimum diameter of the coarsest aggregate fraction, 3/8-in. to No. 4, we obtain a theoretical maximum diameter for the next smaller fraction

$$0.14 \times 0.187 = 0.0262 \text{ in.}$$

14. Based on these results, a quartz-chert aggregate (0.0059 to 0.0234 in.) supplied by the resin manufacturer for use with their epoxy resins was considered suitable to fill the voids between the larger aggregate particles. The percentage of the fine sand in the total aggregate mixture was subsequently varied and the loose unit weight of each aggregate composition determined in order to pinpoint a maximum aggregate bulk density. Plate 1 shows that a minimum void content occurred at about 35% (by weight) fine sand content; this grading was subsequently maintained for all gap-graded mixtures.

15. Warren[25] reported an optimum strength for gap-graded resin mortars when the diameter ratio, the ratio of the mean values of the mesh numbers defining the coarse and the fine sand, was about 1:14 and the percentage of the fine sand ranged between 30 and about 50% of the coarse sand (by weight). This empirical result seems to confirm the validity of the theoretical considerations that led to a diameter ratio of about 1:20 and a weight ratio of nearly 2:1 between coarse and fine aggregates.

16. Additional trial series with limestone aggregate. Crushed limestone aggregate from Tennessee with a continuous grading, as described above, was used instead of siliceous aggregate for a series of trial tests with the polyester resin binder. Since the results of these tests were considered unsatisfactory (see paragraph 37), limestone aggregate was later abandoned in the beam test series.

Resin concrete mixture proportioning

17. Using each quartz-chert aggregate mixture, gap-graded and continuously graded, a series of mixtures was made with both resins, varying the resin content between 10 and 20% with respect to the total aggregate weight. Prismatic and cylindrical test specimens were fabricated from each mixture, and the unit weight, compressive strength, E , and modulus of rupture were determined after 7 days of curing at room temperature. Plates 2 and 3 and tables 2 and 3 summarize the results of these tests. A similar series was subsequently made with the polyester resin and a

continuously graded limestone aggregate; the results are shown in table 4 and plate 4.

18. Two mixtures were finally selected on the basis of strength, shrinkage characteristics, workability, and economy. It was decided that a gap-graded mixture with a 16% epoxy resin content and a continuously graded mixture with a 10% polyester resin content would be used for the main test series. The decision to use a gap-graded aggregate in connection with the epoxy resin and a continuously graded aggregate for the polyester concrete was prompted by the different viscosities of the two resins, which resulted in distinctly different workability and bleeding characteristics. One limestone aggregate-polyester resin mixture (12% resin content) that showed excellent strength characteristics but seemed to shrink excessively was also subjected to further testing.

19. All resin concretes were mixed in a 1 cu ft vertical mixer with additional hand mixing to ensure thorough homogenization. After mixing, the resin concrete was placed into the molds in 1-1/2- to 2-in.-thick layers and compacted with regular tamping rods (polyester concretes) or mechanical tampers (epoxy concretes). Laboratory temperatures during the mixing and placing, as well as during the subsequent curing and testing period, stayed between 70 and 90 F with the relative humidity ranging between 50 and 90%. The lack of close humidity and temperature control in the laboratory is believed to have caused some of the variations in the test results.

Portland cement concrete data

20. Unintentionally, two different portland cement concrete mixtures were used in the epoxy and the polyester resin concrete beam series. Both concretes were proportioned with 3/8-in. maximum size crushed limestone aggregate to have a slump of 2 ± 1/2 in. and compressive strengths of approximately 4000 and 3000 psi, respectively, at 28 days. Mixture data and physical properties of the two concretes are compiled in table 5. It is felt that the use of two different portland cement concretes in the two test series, although unintentional, did not invalidate a comparison of results in the two series since all beams (except 5A) were extremely underreinforced and beam failures were dictated by the tensile strength of the

reinforcement and of the resin concrete layer rather than by the strength of the portland cement concrete itself.

Properties of reinforcing materials

21. The following reinforcing materials were used in the two composite beam series.

a. Deformed high-strength No. 4 steel bars (nominal 1/2-in. diameter), obtained in Mississippi, were used as longitudinal reinforcement. The steel had a yield strength (f_y) of 53,500 psi (plate 5), an f_u of 69,000 psi, and an E of 29.9×10^6 psi. Its stress-strain curve up to 10,000 microstrain* was essentially bilinearly elastoplastic.

b. Stirrups were undeformed No. 2 steel bars, also obtained in Mississippi (f_y = 43,000 psi, f_u = 76,500 psi, E = 29.3×10^6 psi).

c. One-half-in.-diameter deformed polyester-fiberglass rods were also tested. These rods had an E of 6.7×10^6 psi, a linear stress-strain curve up to failure, and an f_u in excess of 87,000 psi (used in beam 5A).

Fabrication of Test Beams

22. A total of fourteen 78-in.-long beams with rectangular cross sections (4 by 9 in.) were cast during the resin concrete composite beam program. In casting the beam in an inverted position, the conventional concrete was first placed and consolidated with internal vibrators. The forms were removed after two days, and moist curing was continued to a concrete age of 7 days, whereupon the specimens were stored in the laboratory air. Preceding the application of the resin concrete layers at 21 days age, all concrete and reinforcement surfaces to be in contact with the resin concrete were first sandblasted and then painted with pure resin. Finally the resin concrete was placed in 1-1/2- or 3-in.-thick

* 10^{-6} in./in.

layers and compacted with tamping rods or mechanical tampers, as mentioned earlier. The total number of beams were fabricated in one ten-beam series and in one four-beam series.

23. This first series (Series A) consisted of the following beams.

a. Beam 1A (reference beam). This beam was a conventional concrete beam with two regular No. 4 deformed steel reinforcing bars and vertical ties (No. 2 bars at 4-in. spacing, except for a 16-in.-wide portion in the beam center that contained no ties). The reinforcement arrangement and exact beam dimensions are shown in table 1 and plate 6 for all beams.

b. Beam 2A. This composite beam had 7-1/2 in. of portland cement concrete and a 1-1/2-in.-thick bottom layer of epoxy resin concrete. The beam was reinforced with steel bars and vertical stirrups as in beam 1A.

c. Beam 3A. A composite beam similar to beam 2A, except that beam 3A had only one reinforcing bar.

d. Beam 4A. Similar to beams 2A and 3A, except that beam 4A did not contain any longitudinal reinforcement.

e. Beam 5A. The same as beam 2A except that this beam was reinforced with two No. 4 deformed fiberglass rods instead of regular reinforcing bars.

f. Beam 6A. A composite beam having 6 in. of portland cement concrete and 3 in. of epoxy resin concrete on the bottom. The beam was reinforced with two steel bars and vertical stirrups identical with those used in beam 2A.

g. Beam 7A. The same as beam 3A except that beam 7A had a 3-in.-thick layer of epoxy concrete in the tension zone.

h. Beam 8A. The same as beam 4A except that beam 8A was cast with a 3-in.-thick layer of epoxy concrete.

i. Beam 9A. The same as beam 4A except that beam 9A contained no shear reinforcement.

j. Beam 11A. The same as beam 8A except that beam 11A contained no stirrups.

24. Due to disappointing results in the polyester concrete pretest series and to limited funding, only four beams were cast and tested in the polyester resin concrete phase of the composite beam program. This series, Series B, consisted of the following beams.

- a. Beam 1B (reference beam). The same as beam 1A of epoxy concrete series.
- b. Beam 6B. The same as beam 6A of epoxy concrete series except that polyester resin, quartz-chert aggregate concrete was used instead of epoxy concrete.
- c. Beam 7B. The same as beam 7A (using polyester concrete instead of epoxy concrete).
- d. Beam 8B. The same as beam 8A (using polyester concrete instead of epoxy concrete).

Two additional beams, identical with beams 2A and 3A of the epoxy resin concrete series, were fabricated using the polyester resin-limestone aggregate concrete mixture described in table 7 for the 1-1/2-in.-thick resin concrete layer. During setting, however, the polyester-limestone concrete developed numerous shrinkage cracks (photograph 5), and the beams were not tested.

Test Procedures

Tests on resin concrete mixtures

25. The following series of preliminary tests was performed on specially prepared specimens.

- a. Modulus of rupture and flexural elastic modulus (7-day tests). To determine the modulus of rupture and the flexural elastic modulus, two 2- by 2- by 11-1/4-in. prisms (photograph 1) made from each mixture were put on roller supports 10 in. apart with their finished side up and third-point loaded at a rate of approximately 0.05 in./min. Dial gages measured midspan deflection under increasing loads (photograph 2). Basic linear elastic equations were used to calculate the elastic modulus and the modulus of rupture.

b. Compressive strength at 7 days. For these tests 3- by 6-in. cylinders, 2-in. cubes sawed from the remains of the prismatic specimens after completion of the flexural tests, or both, served to determine the compressive strength of all trial mixtures at 7 days.

c. Unit weight. All test specimens were weighed and measured before loading to determine their unit weight.

d. Visual bleeding and shrinkage observations. After fabrication, the specimens were repeatedly observed for bleeding and for development of visual gaps between the specimens and the molds that would indicate excessive shrinkage.

26. After selecting the most suitable resin concrete mixtures, a more comprehensive test program was conducted on these mixtures to determine the values of their most important engineering properties.

a. Tensile splitting test. A group of 3- by 6-in. cylinders served to obtain the tensile splitting strength of selected resin concretes in accordance with method CRD-C 77-61 (26).

b. Direct tension test. Using 2- by 6- by 1/4-in. plates as inserts in regular 2- by 2- by 11-1/4-in. prism molds, necked specimens were obtained on which direct tension tests were performed at 7 days age (photograph 3). These specimens were instrumented with 1-in. strain gages to obtain stress-strain curves and Poisson's ratios in tension. The loading rate was about 0.05 in./min.

c. Stress-strain curves. In addition to stress-strain curves and Poisson's ratios in tension, as described above, regular compressional stress-strain curves and Poisson's ratios were determined on strain-gaged 3- by 6-in. cylinders also at 7 days age. The loading rate for these tests was again approximately 0.05 in./min.

d. Shrinkage characteristics. Early shrinkage characteristics of resin concretes were measured with 8-in. mechanical strain gages by inserting the measuring discs into the surface of 2- by 2- by 11-1/4-in. prisms and taking continuous readings

as soon as the resin had set up enough to allow such measurements.

<u>e</u>. <u>Strength development with time.</u> Using 3- by 6-in. cylinders, the change in the compressive strength with time was determined for all three resin concrete mixtures.

27. The length changes of two 2- by 2- by 11-1/4-in. prisms during five temperature cycles between 75 and 150 F (leaving the specimens exposed to each temperature until no further changes were observed) were measured with 6-in. mechanical strain gages.

28. Thermocouples in the center of 1-qt vacuum bottles filled with fresh resin concrete (which was rodded for compaction) served to obtain temperature rise curves for the three mixtures.

<u>Beam tests</u>

29. <u>Loading apparatus.</u> A rigid steel testing frame, shown in photograph 4, was used for all beam tests. The beams were supported on a full rocker system on one side and a half rocker system on the other side, providing a span of 6 ft. Third-point loads were applied by two calibrated hydraulic jacks resting on ball bearings. One-inch-wide pads between the rollers and the beams served to distribute loads and support reactions.

30. <u>Test measurements and instrumentation.</u> Longitudinal strains in the tensile reinforcement were measured in the center of each beam by 1/4-in. resistance strain gages glued to the reinforcing bars. Concrete surface strains at the top and the sides of the beam center were measured with a 2-in.-long mechanical strain gage (for the location of surface strain measurements see plate 7). One-in. dial gages mounted on an independent scaffold, thus unaffected by deformations of the testing frame, measured beam deflections at five points along the span. A hydraulic system consisting of two 20-ton jacks, a control panel, and a 2500-psi precision pressure gage (calibrated before and after the test series) was used to apply and measure loads.

31. <u>Test procedure.</u> In placing the beam in the testing frame, particular attention was given to the exact alignment of the test beam, the supports, and the loading assembly to ensure true axisymmetric bending. The five dial gages, mounted on a separate scaffold, were zeroed against

the underside of the beam. Strain gages were connected with the Wheatstone Bridge, and initial (electrical and mechanical) strain readings were taken.

32. Loads were applied in 500-lb increments (total load), and beam deflections were read after each increase in load. At intervals of 1000 lb (total load) a full set of mechanical, concrete surface strain and electrical reinforcement strain readings were taken. Upon completion of the strain measurements, deflections were read for a second time under the same load, generally about 3 min after the first reading.

33. Loads were completely released at intervals of 3000 lb to check nonelastic deformations (deflections and strains) of the beam prior to the continuation of loading. A full load-unload cycle, leading to a total load 3000 lb higher than the maximum load achieved in the previous cycle, usually took about 15 to 20 min to complete.

34. Cracks were observed throughout the test and all hairline cracks were marked with ink. Upon any significant change in the crack pattern a photograph was taken. All beams were tested to failure.

PART III: TEST RESULTS

Resin Concretes

Results of preliminary tests to select optimum resin concrete mixtures

35. Tests concerning the selection of aggregate type and grading were described in paragraphs 11-16; results are shown in plate 1. Two types of aggregates, a quartz-chert sand and gravel aggregate and a crushed limestone aggregate, with identical gradings were tried during the polyester resin concrete mixture proportioning series. It is interesting to note that the limestone aggregate gave a considerably higher strength than the equivalent quartz-chert aggregate mixtures (tables 3 and 4). However, short pot life, rapid setting, and excessive shrinkage of the polyester-limestone concrete prevented its successful use in the main beam test series. (For more details see below and paragraph 62.)

Trial mixtures

36. Epoxy resin concretes. Results of two series of trial mixtures (using a continuously graded and a gap-graded quartz-chert aggregate and varying the epoxy resin content between 10 and 20% of the total aggregate weight) are summarized in table 2 and plotted in plate 2. Based on the results of these trial mixtures, which indicated a better workability and a somewhat higher strength for gap-graded aggregate mixtures, a 16% resin concrete with gap-graded aggregate was finally selected for the main epoxy concrete beam test series.

37. Polyester resin concretes. As mentioned earlier, two different types of aggregates, i.e. quartz-chert sand and gravel and crushed limestone, were investigated during the polyester resin concrete trial mixture tests. Results of two series with quartz-chert aggregate (again using a gap-graded and a continuously graded aggregate and varying the resin content between 10 and 16%) are compiled in table 3 and plotted in plate 3, while the results for the third series, using continuously graded limestone aggregate, are summarized in table 4 and plate 4. From these tables and plates it can be seen that the limestone aggregate series showed a

considerably higher 7-day strength than both quartz-chert aggregate series. However, all limestone mixtures also exhibited short pot life, flash setting, and excessive shrinkage. Additional series on limestone aggregate-polyester resin concrete mixtures were subsequently conducted, reducing the catalyst content from 1 to 1/2, 1/4, and 1/8% in an attempt to eliminate the undesirable flash setting and reduce shrinkage. However, even a drastic reduction in the catalyst did not satisfactorily eliminate the problems. Precooling of the aggregate and the resin provided a somewhat longer pot life; however, the setting was still very rapid, resulting in high concrete temperatures and excessive shrinkage.

Engineering properties of the selected resin concrete mixtures

38. Summarized in tables 6 to 8 are the following: unit weights, 7-day compressive strengths of 3- by 6-in. cylinders and 2-in. cubes, moduli of rupture of third-point loaded 2- by 2- by 11-1/4-in. prisms, tensile splitting and direct tensile strengths, elastic moduli in tension and compression, 7-day shrinkage values, average coefficients of thermal expansion (between 75 and 150 F), and approximate pot life of the selected epoxy resin concrete and of the two polyester resin concretes (a continuously graded quartz-chert aggregate mixture with 10% resin content and an identically graded crushed limestone aggregate mixture with 12% resin content). Average stress-strain curves in tension and compression for the three resin concretes are shown in plate 8. Plate 9 and table 9 depict the strength-time relations, plate 10 the shrinkage curves, and plate 11 the exothermal temperature rise for the three mixtures.

Results of Beam Tests with Epoxy Resin Concrete

39. Results of the 10 beam tests in Series A are summarized in table 10, which indicates the cross-sectional geometry of individual beams (for exact dimensions see table 1) and lists their calculated and measured ultimate loads, cracking loads, and midspan deflections under various load levels.

Cracking and failure of beams

40. Beam 1A (reference beam). The first hairline cracks in the portland cement concrete appeared at about 4000-lb total load and increased in size and number as the load increased (photograph 6). At about 12,800-lb total load the reinforcement began to yield. This caused rapidly increasing beam deflections and led finally to a compressional failure of the concrete at the top of the beam at an ultimate load of 13,000 lb.

41. Beam 2A. The first hairline cracks in the portland cement concrete appeared at the 6500-lb total load; they did not, however, extend into the epoxy resin concrete layer (photograph 7). Beam deflections at all load levels were considerably smaller than those of the equivalently reinforced reference beam. Increasing loads subsequently caused an increase in the number and size of the portland cement concrete cracks, but the epoxy concrete layer in which the tensile reinforcement was embedded remained uncracked up to a total load of 15,000 lb. At this load the 1-1/2-in.-thick resin concrete layer suddenly developed a single major crack and the steel reinforcement started to yield, causing rapidly increasing beam deflections. Compressional concrete failure finally occurred at an ultimate load of 15,300 lb.

42. Beam 3A. Here the first hairline cracks in the concrete were observed at 3000 lb (photograph 8). Again the epoxy concrete layer remained uncracked while a total of nine cracks gradually developed in the portland cement concrete as loads increased. At 8100-lb total load the epoxy concrete layer failed in tension. Rapid yielding of the single reinforcing bar resulted in compressional concrete failure.

43. Beam 4A. As might be expected of this unreinforced beam, sudden failure occurred due to simultaneous cracking of the tensile (epoxy and portland cement) concrete layer and was not preceded by the formation of visible hairline cracks in the portland cement concrete (photograph 9). However, this mode of failure was distinctly different from that of the other unreinforced beams, which did develop tensile cracks in the portland cement concrete long before the cracking of the resin concrete led to sudden failure.

44. Beam 5A (fiberglass-reinforced). The first visible cracks under

the 6000-lb total load extended from the bottom of the beam through the epoxy concrete layer up into the top third of the beam. Subsequent increases in the load caused a gradual increase in the size and number of cracks (photograph 10) until at 16,600 lb the beam suddenly failed in a compressional mode.

45. Beam 6A. Cracks were first observed under the 8500-lb load. They subsequently became larger and more numerous, but did not start to extend into the epoxy concrete layer until the total load reached 12,000 lb (photograph 11). At 14,000 lb, the reinforcement began to yield, and compressional concrete failure occurred at 14,500 lb.

46. Beam 7A. Hairline cracks started to appear at 5500 lb, and the first crack in the 3-in.-thick epoxy concrete layer was observed under 8000-lb total load (photograph 12). Under the 9000-lb load the reinforcement started to yield, causing a rapid increase in deflection without further increases in load.

47. Beam 8A. Photograph 13 shows a number of hairline cracks that formed in the portland cement concrete under 4000-lb total load while the epoxy concrete layer remained intact. With increasing loads, the cracks in the portland cement concrete increased in size and number until at 5500 lb one of the cracks finally propagated into the epoxy concrete layer, causing sudden failure.

48. Beam 9A. Similar to the preceding test, the portland cement concrete developed a total of six cracks under a load of 3000 lb (photograph 14), thereby transferring tensile forces to the 1-1/2-in.-thick layer of epoxy concrete. The epoxy concrete layer remained intact until 3300 lb when one of the concrete cracks propagated into that layer, initiating sudden failure.

49. Beam 11A. Again, hairline cracks in the portland cement concrete started to form at a relatively low load (about 2500 lb) and became larger and more numerous as the load increased (photograph 15); however, not until a total load of 4200 lb did one of the cracks propagate into the epoxy concrete layer, causing failure.

Beam deflections

50. Load-deflection curves for all 10 beams of the epoxy concrete

series are presented in plate 12. Individual deflection readings at five points along the span are compiled in table 11, and midspan deflections for each beam during all loading and unloading cycles are plotted in plate 13.

Strain readings

51. Average readings, under various increments of load, from two 1-in. bonded resistance wire gages cemented to the reinforcing bar(s) and waterproofed are shown in plate 14. The same plate also summarizes average mechanical strain measurements on the concrete surface, namely, compressive strain measurements on the top beam surface (average of four readings at locations 1, 2, 3, and 4 in plate 7); lateral measurements on the same top surface (average of three readings at locations 5, 6, and 7 in plate 7); and tensile strain measurements at two different elevations in the lower portion of the lateral beam surfaces (i.e. average of four readings at the bottom of the beam, locations 10, 11, 14, and 15 in plate 7); and average of another four readings about 1 in. above the bottom of the beam, locations 8, 9, 12, and 13 in plate 7. It must be emphasized that in the case of the last two mechanical tensile strain measurements the average plotted in plate 14 was repeatedly obtained from widely varying individual readings, since some of the 2-in.-long measuring distances included a crack while others did not. Obviously, the strain readings in cracked sections were very high, while the neighboring uncracked sections hardly showed any strain at all following the formation of a crack in the adjoining section. In some instances uncracked sections close to a major crack showed small compressive strains (up to a maximum of 150 microstrains (10^{-6} in./in.)). The average mechanical tensile strain measurements therefore represent an average strain over a 4-in.-long distance in the beam center that at higher loads frequently included at least one visible crack.

Moment-curvature relation

52. Midbeam curvatures under different loads were calculated from the average compressional strain reading taken on the top beam surface, from two or three different tensile strain readings, i.e. electrical strain readings on reinforcing bars, and from mechanical strain readings at two

elevations on the lateral beam surface. Two or three independent curvature values were thus obtained for each beam and loading condition (table 12). The agreement between the three values was usually surprisingly good, with most individual values staying within less than ±10% of their common mean. Only for very small curvature values (after unloading) and in some instances at very large curvature values (preceding failure) did individual values deviate considerably more than 10% from their mean--except beam 3A where a significant difference between electrical and mechanical strain readings occurred throughout most of the test.

53. To facilitate comparison, average moment-curvature curves (omitting all unloading phases) for the 10 beams of the epoxy concrete series are presented in plate 15a.

Results of Beam Tests with Polyester Resin Concretes

54. It was originally planned to duplicate the whole epoxy concrete beam series in the polyester resin concrete series; however, due to disappointing results with the polyester resin concrete and limited funding, it was decided to curtail the polyester concrete beam program. The results of the four beam tests in the abbreviated polyester resin concrete series, Series B, are summarized in table 13, which lists the cross-sectional geometry of individual beams, calculated and measured ultimate loads, cracking loads, and midspan deflections under various load levels. Two additional beams, with cross sections identical with those of beams 2A and 3A in the epoxy concrete series, were cast using the limestone aggregate polyester resin mixture described in paragraph 18 for the resin concrete layer. However, during setting, numerous large cracks developed in the resin concrete layer (photograph 5) and the two beams were not tested.

Cracking and failure of beams

55. Beam 1B (reference beam). The first hairline cracks were observed at a 3000-lb total load and increased in size and number until at 12,000-lb total load the reinforcement started to yield, causing a compressional failure of the top concrete at 12,280 lb (photograph 16). The cracking and ultimate moment at a somewhat lower loading obtained on this

beam, as compared with reference beam 1A for the epoxy series, was caused by a somewhat smaller beam width (3.90 in. versus 3.97 in.) and a lower compressive strength for the polyester resin beam series.

56. Beam 6B. Cracks in the polyester concrete layer appeared at 9000 lb (photograph 17). At a 10,000-lb total load, the portland cement concrete started to crack. The reinforcement began yielding at about 11,600 lb, leading to compressional concrete failure at the 12,000-lb total load.

57. Beam 7B. A midspan crack in the 3-in. polyester resin concrete layer and in the lower part of the portland cement concrete appeared at 6000-lb total load (photograph 18). Subsequent increases in loading caused a rapid growth of this midspan crack and the formation of several other cracks. At about 6400 lb the reinforcement began to yield and the beam reached its ultimate load-carrying capacity and failed in compression at 6840 lb.

58. Beam 8B. No visible cracks appeared in this unreinforced beam prior to its sudden failure under 3580-lb total load (photograph 19).

Beam deflections

59. The load-midspan deflection curves for the four tested beams of the polyester concrete series are presented in plate 16 and are plotted individually in plate 17. Table 14 summarizes all deflection measurements.

Strain readings and moment-curvature relations

60. Average electrical and mechanical strain readings and computed moment-curvature relations for three of the beams of this series are compiled in table 15 and plates 18 and 15b in the same manner as for the beams in the epoxy concrete series. Reference beam 1B exhibited a drastic difference between electrical strain readings on the reinforcement rods and mechanical strain readings at the concrete surface, possibly caused by an early hairline crack in the measuring distance.

PART IV: DISCUSSION OF TEST RESULTS

Resin Concretes

61. The two binder systems used were arbitrarily chosen as fairly representative examples of a polysulfide-epoxy and a rigid polyester resin system. It must be realized that other epoxy or polyester resin-hardener systems will perform differently. For this reason, the results of a limited investigation should not be generalized as some of the problems encountered in a pilot study could certainly be overcome by a systematic investigation of various resin systems and subsequent selection of the most suitable binder for any particular application.

62. Nonetheless, a few findings in this study appear to be of general importance, particularly those involving some negative results. The results, for instance, indicate clearly that an evaluation of binder systems (as well as of aggregate and mixture compositions) that is entirely based on routine strength tests of standard small laboratory specimens (such as conventional compressive, flexural, and tensile tests) can lead to entirely erroneous conclusions as to the suitability of a particular resin concrete mixture. Besides not reflecting actual field conditions that may rather drastically affect the performance of polymer binders,[5,27] these tests may also overlook another factor of potentially great importance, namely, the effect of specimen size. A small laboratory strength specimen usually will be relatively unaffected by exothermal heat release and volume changes that may lead to very serious problems in actual construction, sometimes making a mixture with excellent laboratory strength results entirely useless for practical applications. The practical significance of strength data obtained from small laboratory specimens under closely controlled conditions is always debatable; however, with resin concrete this appears to be a truly critical question. For instance, excellent strength results were obtained during this investigation on small specimens of a polyester resin limestone aggregate mixture; yet when the same mixture was used in the main beam program the resin concrete developed large cracks and turned out to be entirely unsuitable for the contemplated

application. The problem was caused by the excessive shrinkage of this mixture. This condition had been anticipated in this particular case because unrestrained shrinkage measurements had been made together with the routine strength tests. However, frequently such shrinkage measurements will not be made; and, based on excellent strength results alone, the erroneous conclusion could then be drawn that a very suitable mixture had been developed. Similar problems can be encountered with exothermic heat, humidity effects, coefficient of thermal expansion, and other effects whose importance is governed by shape, size, and boundary conditions of the resin concrete body.

63. The results also show that even under laboratory conditions with closely controlled manufacturing and testing procedures and only moderate environmental changes (the laboratory rooms used were not climate controlled and had temperature variations between 70 and 90 F and the relative humidity ranging between 50 and 90%) a rather large variation in test results can occur. This seems to indicate the sensitivity of resin binders to environmental factors.

Aggregates

64. The tests to develop optimum aggregate gradings and their results were described and discussed in paragraphs 11-15. Two different aggregates, a well-rounded quartz-chert and crushed limestone with identical grading, were used during the polyester resin mixture design series. As emphasized before, the polyester resin limestone aggregate concretes developed a considerably higher strength than all polyester resin quartz-chert aggregate mixtures. However, the pot life was shorter and the temperature rise and shrinkage considerably greater with the limestone aggregate. The author theorizes that this phenomenon was caused, at least in part, by the lower diffusivity of limestone as compared to quartz-chert, which resulted in a faster and higher temperature rise bringing about a quicker and more complete polymerization (as well as greater thermal shrinkage) for the limestone aggregate polyester resin concrete. However, other factors such as particle shape, surface texture, specific surface,

etc., may have been contributing factors. There is also a possibility of a chemical reaction between the limestone aggregate and the polyester resin[1] (which has a pH of about 3.8). However, a brief additional test series, in which 100 g of polyester resin and 1 g of 60% methyl-ethyl ketone peroxide diluted in dimethyl phthalate were mixed with 50 g of both the quartz-chert and the limestone aggregate that passed the No. 200 sieve and the temperature rise of these two mixtures was compared with the temperature rise of the pure resin-hardener system, failed to indicate any such reaction (plate 19). The temperature measurements showed only a minute difference between the limestone and the quartz-chert aggregate which can be explained by the different diffusivity of the two materials. The author strongly feels that the observed phenomenon warrants further investigation; however, the scope of this program unfortunately did not allow a systematic study. It might be mentioned here that Stamenov, Goudev, and Malcev[19] have also reported higher strength results for polyester concretes using a basic rock (diabase) rather than quartz as fine aggregate. J. Michie* reported excellent strength results for a polyester resin concrete with basalt aggregate used in a Nevada field test.

Resin Content

65. Following the selection of resin binder systems, aggregates, and aggregate gradings, the next step was the development of optimum mixture proportions. Tables 2-4 and plates 2-4 show the results of the trial series with varying amounts of epoxy or polyester resins.

Epoxy concrete series

66. Both the gap-graded and the continuously graded quartz-chert aggregate series reached a maximum unit weight at a resin content of about 14 to 16% of the total aggregate weight. This result is consistent with theoretical considerations (it being roughly the amount of resin necessary to fill the 21 and 24% voids between the densely packed aggregate particles), and with results of earlier investigations.[25] The modulus of

* Private communication with Mr. J. Michie, Southwest Research Institute, Oct 1966.

rupture for both series increased with increasing resin content, which, too, is a normal result for resin concretes.[6,14,19] The range of modulus of rupture values between 1795 and 2995 psi can be considered typical for epoxy concretes,[6,9,15] as can the modulus of rupture for the pure resin (5395 psi). Somewhat different results were obtained for the two series with regard to compressive strength. While the gap-graded series showed a distinct increase in compressive strength with increasing resin content throughout the tested range, the continuously graded series developed a strength maximum at 14% resin content with values for higher and lower resin contents falling considerably below that maximum. Several earlier investigators[1,3,15,27] have reported that the compressive strength of resin concretes reached a maximum with a resin content of about 15 to 20%, depending on the type of binder and aggregate used. Bares[3] suggested recently that two maxima for the compressive strength as a function of the resin content occur in resin concretes, one at a very high resin content (approaching the case of pure resin) and another at a resin content of the above-indicated magnitude. It may thus be that the gap-graded mixture would have reached its first maximum slightly above 20% resin content (by total aggregate weight). The compressive strength of the pure resin was 12,500 psi, almost identical with the highest value obtained with the gap-graded aggregate at a resin content of 20%. The range of compressive strength values determined for the epoxy resin trial mixtures, in general, corresponds to those usually found in the literature for resin concretes,[6,9,15] though occasionally much higher values have been reported.[28]

67. In his comments in reference 25, Bares also presents a plot showing the effect of resin content upon the Young's modulus of resin concretes. His diagram indicates that for an epoxy resin concrete a maximum elastic modulus was obtained with a resin content slightly less than that necessary to produce a maximum compressive strength, i.e. about 14% resin content with respect to the total aggregate weight. While this was found to be true in this study for the continuously graded aggregate, the flexural elastic modulus of the epoxy resin concrete with gap-graded aggregate stayed nearly the same for various resin contents (around

1×10^6 psi) except for the mixture with 20% resin content, which developed a considerably higher modulus (1.6×10^6 psi). The magnitude of values in general corresponds to those cited by Bares for epoxy resin concretes. However, it should be emphasized that the method used to determine the flexural elastic modulus in the trial series of this program is rather crude and cannot be expected to yield very accurate results.

68. A few remarks remain to be made about the effect of the resin content upon the workability of the tested epoxy concrete mixtures. It seems that an optimum workability was obtained with a resin content of about 14 to 16%. Lower resin contents resulted in dry, harsh mixtures with poor workability; higher resin contents tended to cause serious bleeding.

Polyester resin concretes

69. Within the tested range of resin contents (10 to 20%), the concrete unit weight tended to decline with increasing resin content in all three test series (i.e. gap-graded quartz-chert, continuously graded quartz-chert, and limestone aggregate). Due to the lower density of the polyester resin (approximately 1.03 g/cu cm) a resin content of roughly 10% (continuously graded) and 12% (gap-graded) theoretically sufficed to fill the voids between the densely packed aggregate particles. The lower viscosity of the polyester resin, as compared with the epoxy, made it easier to approach a 100% compaction with a low resin content.

70. The modulus of rupture of the mixtures with gap-graded quartz-chert aggregate stayed around 1900 psi for all three resin contents tested in this series. Both continuously graded aggregate series exhibited a maximum modulus of rupture with 12% resin content, the values for the limestone aggregate concretes being considerably higher than those for the quartz-chert aggregate concretes (e.g. 2316 psi versus 1703 psi for 12% resin content). Practically all polyester resin concrete mixtures in the trial series, however, developed a lower flexural strength than comparable epoxy resin mixtures. The two continuously graded aggregate series also showed a compressive strength maximum at 14% resin content, while the compressive strength of the gap-graded aggregate series was not clearly affected by the resin content. Again, the strength values for the limestone aggregate

mixtures were much higher than those for the two quartz-chert aggregate series. In the majority of cases they even exceeded the compressive strength obtained on equivalent epoxy resin concrete mixtures.

71. The observation that polyester resin concretes can develop a higher compressive strength than epoxy concretes and at the same time be lower in flexural strength was also made by Bares.[6] In turn, Cirodde[15] reported that the opposite can also be true. L'Hermite,[28] in a very recent paper on high-strength resin concretes (he reported compressive strengths up to above 20,000 psi for epoxy resin concretes and an average compressive strength of 12,000 psi for polyester concretes), gives almost the same ratio between tensile and compressive strengths for both types of resin concretes (i.e. about 0.125 to 0.14).

72. The magnitude of values for the compressive strength and modulus of rupture obtained in all three polyester resin concrete series was similar to those indicated by most other investigators;[6,25,29,Varnell*] however, Stamenov, et al.,[19] reported compressive strength values up to about 17,000 psi and a modulus of rupture as high as 5200 psi for a polyester resin with a diabase and quartz-chert aggregate mixture with approximately 19% resin content (with respect to aggregate weight).

73. Finally, Young's modulus in flexure for the two continuously graded aggregate series declined with increasing resin content, and again the limestone aggregate polyester resin concretes developed much higher moduli than their counterpart quartz-chert aggregate mixtures (e.g., 1.45×10^6 versus 1.14×10^6 psi at 12% resin content). The flexural elastic modulus for the gap-graded quartz-chert aggregate series stayed at a constant 1.06×10^6 psi for the three resin contents tested. Since rigid polyester resins usually have a higher elastic modulus than epoxy resins,[1,30] polyester resin concretes are normally expected to show a higher elastic modulus than epoxy concretes.[25] While this was found to be true in this investigation as far as the secant modulus in compression is concerned, the flexural elastic modulus and the tangent modulus in tension of the quartz-chert aggregate concretes did not change much when

* Personal communication with Mr. W. R. Varnell, Concrete Development Corporation, San Antonio, Tex., 26 Oct 1966.

polyester resin was used instead of epoxy resin. Due to the high exotherm of the polyester resin, an attempt to cast 2- by 2- by 11-1/4-in. prismatic and 3- by 6-in. cylindrical specimens of pure polyester resin failed and the intended comparison of their properties with those of similar pure epoxy resin specimens could not be made.

74. Cast polyester resins are known for their tendency to shrink significantly during polymerization[1,18,31,Varnell*] (a phenomenon that usually poses no problems with epoxy resins[18,32]). Such shrinkage was indicated during the trial series by the development of small gaps between the molds and the polyester concrete specimens during the setting process. Particularly drastic shrinkage could be observed in the limestone aggregate series and in all series the shrinkage appeared to increase as the resin content increased.

75. The workability of the continuously graded quartz-chert aggregate polyester resin mixture series reached an optimum at approximately 10 to 12% resin content, and that of the continuously graded limestone aggregate and the gap-graded quartz-chert aggregate mixtures at about 12 to 14% polyester resin content (of total aggregate weight). Higher resin contents quickly led to excessive bleeding of the concrete, while lower resin contents resulted in harsh mixtures with poor compactability.

76. After the conclusion of the trial series, a continuously graded quartz-chert aggregate mixture with 10% polyester resin content and a crushed limestone aggregate mixture with identical grading and 12% resin content were selected for the beam test program. Engineering properties of these mixtures are summarized in tables 6, 7, and 8 and are discussed below.

Engineering Properties of the Selected Resin Concretes

Compressive strength

77. After 7 days of curing at room temperature, the three selected mixtures developed the following compressive strengths.

* See footnote, page 28.

Mixture	Compressive Strength, psi: 2-in. Cubes	Compressive Strength, psi: 3- by 6-in. Cylinders	Ratio of Cube to Cylinder Strength
Gap-graded quartz-chert aggregate with 16% epoxy resin	10,524 (10,890)*	6,556	1.60
Continuously graded quartz-chert aggregate with 10% polyester resin	6,386 (7,127)*	6,016	1.06
Continuously graded crushed limestone aggregate with 12% polyester resin	13,920 (11,597)*	13,478	1.03

* Values in parentheses indicate results obtained in the trial mixture series.

The ratio of cube to cylinder strength was thus extremely high for the epoxy concretes and very low for the polyester concretes. Presumably this difference is caused by the lower elastic modulus of the epoxy resin, which perhaps makes the epoxy concrete more sensitive to end restraints.

Tensile strength

78. Three methods were used to test the tensile strength of the selected resin concretes: flexural tests (modulus of rupture), tensile splitting tests, and direct tension tests. Their results are compared below:

Mixture	Modulus of Rupture (R), psi	Splitting Strength (S), psi	Direct Tensile Strength (T), psi	Ratios: R/T	S/T	T/CS*
Gap-graded quartz-chert aggregate with 16% epoxy resin	2613 (2700)**	1140	1600	1.63	0.71	0.244
Continuously graded quartz-chert aggregate with 10% polyester resin	1254 (1366)**	1162	630	1.99	1.84	0.10[illegible]
Continuously graded crushed limestone aggregate with 12% polyester resin	2658 (2316)**	2152	1288	2.06	1.67	0.09[illegible]

* CS, cylinder compressive strength.
** Values in parentheses indicate results obtained in trial series.

It is thought that the low tensile splitting strength for the epoxy resin concrete (leading to the unusually low ratio of 0.71 between splitting and direct tensile strength) is also a result of the low elastic modulus, relatively high tensile strength, and pronounced curvature of the stress-strain curve of the epoxy resin concrete. The tensile splitting test is, after all, strictly valid for brittle and fairly linearly elastic materials only; and since the epoxy resin concrete tested was not such a material, it is not surprising that its S/T ratio did not fall within the usual range. For the polyester resin concrete with strength properties more similar to those of ordinary concrete, the ratio resembled that usually found for portland cement concrete.

79. The ratio of direct tensile to compressive strength for the two polyester resin concretes is only about 30 to 50% higher than the ratio that would be expected for a conventional concrete of comparable compressive strength, whereas the epoxy resin concrete developed an extremely high ratio of 0.244, which is about three times as high as that for comparable portland cement concrete.

Elastic modulus and stress-strain curves

80. Stress-strain curves in compression and tension were determined on 7-day-old specimens of all three resin concrete mixtures, and the secant moduli between 0 and 5000 psi in compression (E_c) and between 0 and 1000 psi in tension (E_t) were computed as follows:

Mixture	E_c 10^6 psi	E_t 10^6 psi	E_t/E_c
Gap-graded quartz-chert aggregate with 10% epoxy resin	0.969	1.935	2.0
Continuously graded quartz-chert aggregate with 10% polyester resin	1.50*	2.07**	1.38
Continuously graded crushed limestone aggregate with 12% polyester resin	3.56	3.34	0.94

* 0 to 4000 psi.
** 0 to 600 psi.

Plates 8a and 8b show that the stress-strain curve of the polyester resin limestone aggregate mixture was nearly linear over a wide range of stresses with a sharp curvature close to the ultimate compressional stress, while the stress-strain curve of the epoxy concrete had an appreciable and fairly constant curvature throughout. This explains the marked difference in the E_t/E_c ratios. The shape and position of the stress-strain curve of the polyester resin quartz-chert aggregate concrete falls between the other two. It should also be noted that both the ultimate compressional and tensile strains of the epoxy concrete were much higher than those of both polyester concretes (i.e. 10,400 versus 6000 psi in compression and 1300 versus about 400 psi in tension).

Volume changes

81. Total linear shrinkage curves (after setting) were obtained on 2- by 2- by 11-1/4-in. prisms. It is thought that the total shrinkage consists of:

- a. Thermal volume changes due to thermal contraction during the cooling off period after the hardening of the resin.
- b. Isothermal volume changes covering all volume changes not due to temperature variations, such as volume (or density) changes caused by polymerization.

The total (autogenous) linear shrinkage of the epoxy concrete was less than 1×10^{-4} in./in. or about a third to a fifth of the values normally encountered with portland cement concrete; both polyester concretes developed much higher shrinkage. The selected polyester resin quartz-chert aggregate mixture showed a final shrinkage (after 7 days) of about 9×10^{-4} in./in. and the polyester resin limestone aggregate shrinkage increased to an excessive 49×10^{-4} in./in. Values of the same magnitude were reported by Kreijger[32] for thin films of polyester resins. It is obvious that shrinkage of this magnitude will pose serious problems, especially in composite construction.

82. The question of shrinkage of polyester resin concretes is still controversial. In 1962, Franz and Bossler,[33] and subsequently others,[34,35] reported that "shrinkage values of polyester concretes correspond to those of regular concretes." However, it appears that Franz and Bossler did not

start their shrinkage measurements until the day after the fabrication of their specimen, and consequently measured only a portion of the total shrinkage (see plate 10).

Exothermal heat

83. Temperature rise curves obtained from resin concrete in 1-qt vacuum bottles for the three selected mixtures are shown in plate 11. Due to the relatively low initial mixture temperature (65 to 69 F), two of the three resin concretes were unusually slow to react. Polymerization did not get under way until about 2 hr after mixing for the two mixtures containing quartz-chert aggregate. In the case of the polyester resin concretes, a new shipment of resin was used that may also account for the much slower hardening than observed during all other parts of the program.

84. At any rate plate 11 shows that:

a. Both polyester resin concretes had a considerably steeper temperature rise and a higher peak temperature than the epoxy concrete, indicating a larger and faster release of exothermal heat.

b. The use of limestone aggregate somehow accelerated the polymerization of the polyester resin concrete as evidenced by the shorter pot life, the somewhat steeper slope of the temperature rise curve, and the higher peak temperature.

Linear coefficient of thermal expansion

85. Both polyester resin concretes exhibited rather small coefficients of thermal expansion (6.6 and 7.5 x $10^{-6}/^{o}F$), which were somewhat below the range of 8.3 to 12.0 x $10^{-6}/^{o}F$ reported by Liesegang[34,35] for polyester concrete. Franz and Bossler,[33] however, have mentioned values between 7.2 and 7.8 x $10^{-6}/^{o}F$ for room-cured polyester concretes. The limestone aggregate concrete showed a smaller coefficient than the quartz-chert aggregate concrete, probably due to the lower coefficient of thermal expansion of limestone.

86. Surprisingly, the coefficient of thermal expansion was considerably higher for the epoxy resin concrete (13.2 x $10^{-6}/^{o}F$) than for both polyester resin concretes. Based on the coefficients given in the

literature for pure epoxy and polyester resins,[30,36,37] the opposite would have been expected.

Composite Beam Tests

Epoxy concrete composite beam series

87. The replacement of a 1-1/2- and a 3-in.-thick layer of portland cement concrete on the tension side of unreinforced and reinforced concrete beams by epoxy resin concrete resulted in a distinctly increased ultimate moment, as expected. However, this contribution to the flexural (and probably the shear) strength, which in the case of typically reinforced beams was in the order of 10 to 20%, would hardly justify the additional expense of an epoxy resin concrete layer since the same or a higher increase in strength can normally be realized more easily and cheaply by conventional means, e.g. additional reinforcement, larger cross sections, etc. However, in the case of unreinforced beams a strength increase of about 100 to 200% was realized through the use of epoxy resin concrete layers.

88. Thus, perhaps more significant than its contribution to the strength is the ability of an epoxy resin concrete layer to provide a noncracking, corrosion-resistant, impermeable cover protecting the embedded reinforcement from corrosion even in highly aggressive environments and in situations where ordinary portland cement concrete would have cracked long before the epoxy, exposing the reinforcement to the environment. Whether or not equivalent corrosion protection can also be obtained more cheaply by other means or whether the corrosion protection plus the moderately increased strength and stiffness justify the additional expense of an epoxy concrete layer are questions that though important, cannot be resolved within the scope of this feasibility study.

89. Comparison with analysis and individual results. The cracking loads for all beams with a resin concrete layer were derived in an elastic analysis that assumed an ultimate tensile strain capacity of 1300×10^{-6} in./in. for the epoxy resin concrete (Appendix A). In addition, the yield moment of all reinforced concrete beams was computed using ACI Code[38] equation 16-1 with $\phi = 1$ and disregarding the contribution of the resin

concrete layer, which was considered cracked before yielding of the reinforcement started.

90. For the fiberglass-reinforced beam, 5A, an analysis similar to that described by Sinha and Ferguson,[39] but again setting $\phi = 1$, was used to obtain the ultimate moment.

91. The higher of the two computed moments (i.e. the moment at which the resin concrete cracked or the moment at which the reinforcement yielded or the concrete failed) was taken as the ultimate moment and transformed into the ultimate load-carrying capacity and compared with the measured ultimate load. The agreement between calculated and measured cracking and ultimate loads was fair for the majority of beams; however, three unreinforced and one reinforced beam with epoxy resin concrete layers showed large differences between calculated and measured ultimate loads. It is felt that variations in the resin concrete are responsible for the discrepancy. The individual results are as follows.

- a. <u>Beam 1A (reference beam).</u> As usual, the measured ultimate moment was in very close agreement with the calculated ultimate moment (ACI Code 318-63[38] equation 16-1, using $\phi = 1$), the difference being less than 1%.
- b. <u>Beam 2A.</u> The epoxy resin concrete layer was uncracked up to a total load of 15,000 lb (while the concrete above it showed first visible tensile cracks at 6500 lb) or up to a load about 15% higher than the ultimate load of the reference beam and 33% higher than the computed cracking load. This indicated that the actual tensile strength of the resin concrete layer in this beam was considerably higher than assumed in the analysis. Failure of the beam occurred shortly after the resin concrete cracked at a total load of 15,300 lb, almost 20% above the ultimate load of the reference beam.
- c. <u>Beam 3A.</u> Cracking of the epoxy concrete layer occurred at 8100 lb (the first concrete cracks were observed at 3000 lb), and was presently followed by the failure of the beam. The calculated cracking or ultimate moment was within 3% of the

test result, which was about 20% higher than the theoretical ultimate moment for the same beam without a resin concrete layer.

d. Beam 4A (unreinforced). The calculated ultimate moment was some 12% below the test result, which, in view of the possible variation in the epoxy concrete, was an acceptable agreement. The determined ultimate moment is thus about three times the ultimate moment that would be expected for a plain unreinforced concrete beam of the same cross section (assuming 400-psi tensile strength of the concrete).

e. Beam 5A (fiberglass-reinforced). Both the calculated cracking and ultimate moments were in very close agreement with the test results. Due to the low elastic modulus of the fiberglass reinforcement, the resin concrete layer cracked under rather low loads, comparable to those of unreinforced beam 4A.

f. Beam 6A. Due to a lower strength of the resin concrete, possibly caused by temperature and shrinkage stresses, this beam developed a lower ultimate moment than beam 2A despite its 3-in.-thick epoxy concrete layer. The resin concrete layer cracked under 12,000 lb (calculated cracking load 12,980 lb, or about 8% higher); the first concrete cracks were observed at 8500 lb, and the ultimate load was reached at 14,500 lb, i.e. 12% above the calculated ultimate flexural load. In this case, as in the case of beams 2A and 7A, it is difficult to explain why the measured ultimate moment was much higher than the computed ultimate moment despite the fact that the resin concrete had already cracked at loads significantly below the ultimate. The author suggests as a hypothetical explanation that the excellent bond between the epoxy concrete and the reinforcing steel may have caused a restraint of the transverse contraction of the reinforcing bar at the crack section, resulting in an increased yield strength.

g. Beam 7A. The calculated cracking or ultimate load of 9240 lb was 2.7% above the measured ultimate load (9000 lb) and 15% above the observed cracking load (8000 lb). Since the computed yield load (6800 lb) was much lower than the measured ultimate load and since cracking of the epoxy concrete at 8000 lb did not result in immediate yielding of the reinforcement, the hypothetical explanation mentioned in subparagraph f is again inferred.

h. Beams 8A through 11A. Apparently due to a lower strength of the resin concrete layer, these three unreinforced beams developed only 52 to 74% of their predicted flexural capacity. However, their failure moment was still almost two to three times as high as what was expected for an unreinforced portland cement concrete beam of equivalent dimensions.

92. Deflections and curvature. Since the epoxy resin concrete layer did not crack until the beams approached failure (except beam 5A with fiberglass reinforcement), the cross-sectional moment of inertia remained higher than in conventional reinforced beams; consequently, the curvature and deflections of beams with epoxy resin concrete layers were significantly smaller. Plate 12 and tables 9 and 10 show that the midspan deflections of reference beam 1A were between approximately 20 and 50% higher for any given load than those of beams 2A and 6A, which had 1-1/2- and 3-in. epoxy resin concrete layers. The difference between beam 2A with a 1-1/2-in. layer and beam 6A with a 3-in. layer of epoxy resin concrete was relatively small as far as their deflections were concerned, but was more distinct with respect to curvature (plate 15).

Polyester concrete
composite beam series

93. Due to the lower tensile strength and much lower ultimate tensile strain of the polyester resin quartz-chert aggregate concrete used, the contribution of this concrete layer to the flexural strength of reinforced beams was small. The developed ultimate strength of reinforced beams with 1-1/2- or 3-in.-thick polyester concrete layers was only 4% higher than the strength predicted by the ACI Code for a conventional beam

of equal cross section and reinforcement. Thus, it can be said that the polyester concrete layer contributed very little to the ultimate strength of reinforced beams. However, the layer did increase the cracking load of the composite beams to about twice the cracking load of similar conventional reinforced beams. Consequently, beams with polyester resin concrete layers exhibited somewhat smaller deflections and curvatures in the lower load range than conventional beams, but generally it must be considered that the investigated polyester resin concretes showed little promise for application in this type of composite structure. The total failure experienced when trying to use the polyester resin limestone aggregate concrete (which had shown good tensile strength in routine tests) was described earlier.

PART V: CONCLUSIONS AND RECOMMENDATIONS

94. The results of this investigation showed that resin concrete layers can successfully be used in the tension zone of flexural members to increase their stiffness and strength. For the materials and beam geometry chosen, 1-1/2-in.-thick layers of epoxy resin concrete led to a 10 to 20% increase in the ultimate moment of reinforced beams and up to a 200% increase in the load-carrying capacity of unreinforced beams. Thicker (3-in.) layers did not appear to more beneficially affect the moment capacity, possibly due to increasing internal stresses (shrinkage and temperature) in the thicker layers.

95. More important than its influence on strength is the ability of a properly designed resin concrete layer to provide a noncracking moisture barrier and corrosion protection for the embedded reinforcement. While the conventional concrete above the 1-1/2- and 3-in.-thick epoxy concrete layers developed the usual hairline cracks under relatively moderate loads, the epoxy concrete layer on the bottom of the beam remained uncracked up to or very nearly up to failure, thus providing a reliable built-in vapor barrier and corrosion protection.

96. Although the polyester resin concretes chosen for this program were capable of developing a higher compressive strength than the epoxy resin concretes used, their modulus of rupture, direct tensile strength, and tensile strain capacity fell consistently below those of the epoxy concrete. This, together with the higher exotherm and the excessive autogenous shrinkage, made the investigated polyester concrete unsuitable for the intended purpose. Tests on a few composite beams with polyester resin concrete layers yielded disappointing results.

97. In developing high-strength resin concrete mixtures, it should be kept in mind that routine laboratory strength specimens will not properly reflect the potential deficiencies of the mixture with respect to shrinkage, exotherm, thermal expansion, creep, sensitivity to environmental factors, etc. Separate tests should, therefore, be conducted to evaluate these properties before any mixture can be considered suitable for a particular practical application, regardless of how good its strength

properties may have been in standard laboratory tests.

98. The effect of aggregate mineralogy on the polymerization of resins should also be the subject of further investigation.

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Table 1

Epoxy and Polyester Beam Program

Beam Dimensions

Beam No.	Average Depth in.	Average Width in.	d* in.	Thickness of Resin in.	Type of Reinforcement	Area of Reinforcement sq in.
				Epoxy Resin Series		
1A	9.00	3.97	8.00	--	High-strength steel	2(0.2)
2A	9.00	3.63	8.00	1.35	High-strength steel	2(0.2)
3A	8.91	3.92	8.00	1.42	High-strength steel	0.2
4A	9.12	3.93	--	1.56	None	--
5A	9.06	3.87	8.00	1.69	Fiberglass	2(0.18)
6A	8.99	3.81	8.00	2.56	High-strength steel	2(0.2)
7A	9.05	3.98	8.00	2.79	High-strength steel	0.2
8A	8.99	3.72	--	3.04	None	--
9A	9.02	3.98	--	1.44	None	--
11A	8.99	4.00	--	3.09	None	--
				Polyester Resin Series		
1B	9.1	3.90	8.0	--	High-strength steel	2(0.2)
6B	9.0	3.90	7.60	2.88	High-strength steel	2(0.2)
7B	8.9	3.95	7.90	3.00	High-strength steel	0.2
8B	9.0	3.95	--	3.00	None	--

* Distance from centroid of reinforcement to the top of the beam.

Table 2

Results of Epoxy Resin Concrete Trial Mixtures with Quartz-Chert Aggregate

Mixture No.	Specimen No.	Aggregate Grading	Resin Content % Total Aggregate Weight	Age at Test days	Unit Weight lb/cu in.		Modulus of Rupture, psi		Compressive Strength, psi			Flexural Elastic Modulus, psi, Under 600- to 1200-lb Load	
					Single	Average	Single	Average	Single		Average	Single	Average
A	A	Continuous grading: $A = \left(\frac{d}{D}\right)^{0.4}$	10	7	0.0804	0.0798	1960	1795	5,090	4,740	4,690	--	--
	A'				0.0793		1630		4,870	4,070*		730,000	730,000
B	B		12	7	0.0804	0.0814	2280	2365	6,250	7,290*	7,660	1,140,000	1,140,000
	B'				0.0824		2450		7,620	9,480*		1,140,000	
C	C		14	7	0.0840	0.0840	2220	2515	10,100	9,470	9,840	1,331,000	1,464,000
	C'				0.0840		2810		10,100	9,690		1,597,000	
D	D		16	7	0.0837	0.0840	2410	2410	7,590	7,670	7,675	666,000	713,500
	D'				0.0842		2410		7,950	7,490		761,000	
E	E		18	7	0.0824	0.0825	2440	2530	8,190	8,000	8,270	726,000	726,000
	E'				0.0826		2620		8,410	8,480		726,000	
F	F		20	7	0.0813	0.0812	2960	2995	8,510	8,660	8,410	799,000	844,000
	F'				0.0811		3030		8,200	8,260		889,000	
I	I	Gap grading: 35% 0.0059 to 0.0234 in., 65% 0.187 to 0.375 in.	12	7	0.0820	0.0821	2720	2550	7,740	7,050	7,840	1,140,000	1,235,500
	I'				0.0822		2380		8,150	8,410		1,331,000	
II	II		14	7	0.0826	0.0828	2630	2555	9,920	10,250	9,950	1,000,000	920,500
	II'				0.0831		2480		9,980	9,650		841,000	
III	III		16	7	0.0822	0.0822	2600	2700	10,900	11,070	10,890	998,000	1,113,000
	III'				0.0822		2800		11,200	10,400		1,228,000	
IV	IV		18	7	0.0813	0.0812	2680	2695	10,700	10,400	10,380	939,000	1,002,000
	IV'				0.0811		2710		9,930	10,500		1,065,000	
V	V		20	7	0.0811	0.0809	2830	2885	12,800	12,000	12,550	1,597,000	1,597,000
	V'				0.0808		2940		12,700	12,700		1,597,000	
3	A		100	7	0.0538	0.0539	5200	5395	13,000	12,200	12,500	246,000	254,000
	A'				0.0540		5590		12,500	12,300		262,000	

* 8 days.

Table 3

Results of Polyester Resin Concrete Trial Mixtures with Quartz-Chert Aggregate

Specimen No.	Aggregate Grading	Age at Test days	Resin Content %	Catalyst Content %	Unit Weight lb/cu in. Single	Unit Weight lb/cu in. Average	Modulus of Rupture, psi Single	Modulus of Rupture, psi Average	Compressive Strength, psi Single	Compressive Strength, psi Average	Young's Modulus in Flexure 10^6 psi Under 600- to 1200-lb Load Single	Young's Modulus in Flexure 10^6 psi Under 600- to 1200-lb Load Average
I-C	Continuous grading	7	10	1.5	0.084	0.084	1400	1366	7458	7127	1.331*	1.257
I-D	Continuous grading	7	10	1.5	0.083		1331		6795		1.183*	
II-C	Continuous grading	7	12	1.5	0.081	0.081	1736	1703	7476	7292	1.141	1.141
II-D	Continuous grading	7	12	1.5	0.081		1669		7107		1.141	
III-C	Continuous grading	7	14	1.5	0.083	0.083	1563	1532	7552	7395	0.996	0.968
III-D	Continuous grading	7	14	1.5	0.082		1500		7237		0.940	
IV-C	Continuous grading	7	16	1.5	0.080	0.080	1513	1491	7745	7199	0.726	0.847
IV-D	Continuous grading	7	16	1.5	0.080		1469		6653		0.968*	
1-B	Gap grading	7	12	1.0	0.079		1938		7424		1.065	
2-B	Gap grading	7	14	1.0	0.078		1800		6879		1.065	
3-B	Gap grading	7	16	1.0	0.079		1950		7277		1.065	

* Young's modulus calculated from 600 to 1000 lb.

Table 4

Results of Polyester Resin Concrete Trial Mixtures with Limestone Aggregate

Specimen No.	Aggregate Grading	Age at Test days	Resin Content % Aggregate Weight	Catalyst Content % Resin Weight	Unit Weight lb/cu in.		Modulus of Rupture, psi		Compressive Strength, psi		Flexural Elastic Modulus 10^6 psi Under 600- to 1200-lb Load	
					Single	Average	Single	Average	Single	Average	Single	Average
I-A	Continuous grading	7	8.0	1.0	0.080	0.083	1286	1443	2,950	3,871	0.887*	0.777*
II-B		7	8.0	1.0	0.085		1600		4,792		0.666	
II-C		7	10.0	1.0	0.085	0.086	1931	1935	9,381	11,032	1.597	1.525
II-D		7	10.0	1.0	0.087		1938		12,682		1.452	
III-C		7	12.0	1.0	0.084	0.085	2319	2316	11,978	11,597	1.452	1.452
III-D		7	12.0	1.0	0.086		2313		11,216		1.452	
IV-C		7	14.0	1.0	0.084	0.084	2169	2241	12,933	12,487	1.597	1.597
IV-D		7	14.0	1.0	0.084		2313		12,041		1.597	
V-C		7	16.0	1.0	0.083	0.083	2044	2115	10,452	11,693	1.229	1.341
V-D		7	16.0	1.0	0.083		2186		12,933		1.452	
VI-C		7	18.0	1.0	0.081	0.082	1994	2015	9,254	9,882	1.452	1.392
VI-D		7	18.0	1.0	0.082		2036		10,509		1.331	
VII-C		7	20.0	1.0	0.082	0.082	2081	2041	11,260	11,098	1.228	1.280
VII-D		7	20.0	1.0	0.082		2000		10,935		1.331	

* 600- to 1000-lb load.

Table 5

Portland Cement Concrete Mixture Data and Results of Tests

Material	Volume, cu ft (Solid)	Weight lb
Mixture Data (1-Bag Batch)		
Epoxy Resin Beam Series		
Type I cement	0.479	94
Crushed fine limestone aggregate	2.195	365.1
Crushed coarse limestone aggregate	2.109	354.8
Water	1.217	75.8
Admixture	None	

W/C ratio: 0.806 Slump: 1-1/2 in.
Cement factor: 4.5 bags/cu yd Unit weight freshly mixed: 145 pcf

Compressive strength of 6- by 12-in. cylinders, psi

				Avg
7 days	3040	3000	2820	2953
28 days	4090	4060	4300	4150

Polyester Resin Beam Series

Material	Volume, cu ft (Solid)	Weight lb
Type II cement	0.479	94
Crushed fine limestone aggregate	2.154	358.33
Crushed coarse limestone aggregate	2.070	349.44
Water	1.297	80.8
Admixture	None	

W/C ratio: 0.86 Slump: 2 $\pm$ 1/2 in.
Cement factor: 4.5 bags/cu yd Unit weight freshly mixed: 144 pcf

Compressive strength of 3- by 6-in. cylinders, psi

				Avg
7 days	1768	1768	1888	1908
	1895	2093	2037	
28 days	2709	2560	2723	2830
	3041	3062	2885	

Table 6

Engineering Properties of Selected Epoxy Resin Concrete, Gap Graded Quartz-Chert Aggregate, 16% Epoxy Resin (7-day Tests)

Specimen No.	Specimen Type	Unit Weight lb/cu in. Single	Unit Weight lb/cu in. Average	Compressive Strength, psi Single	Compressive Strength, psi Average	Modulus of Rupture, psi Single	Modulus of Rupture, psi Average	Tensile Splitting Strength, psi Single	Tensile Splitting Strength, psi Average	Direct Tensile Strength, psi Single	Direct Tensile Strength, psi Average	Secant Modulus Compression, psi 0 to 5000 psi Single	Secant Modulus Compression, psi 0 to 5000 psi Average	Secant Modulus Tension, psi 0 to 1000 psi Single	Secant Modulus Tension, psi 0 to 1000 psi Average	% Shrinkage 7 days	Pot Life
C-1	3- by 6-in. cylinders	0.0816		6,827	6,556							947,000	969,000				
C-2		0.0806		6,284								990,000					
P-1'	2-in. cubes	0.0815		10,900	10,524												
P-2'		0.0814		10,147												0.008	Approximately 30 to 60 min
P-1	2- by 2- by 11-1/4-in. prisms	0.0815	0.0814			2581	2613										
P-2		0.0814				2644											
S-1	3- by 6-in. cylinders	0.0819						1280	1140								
S-2		0.0816						1000									
T-1	Necked specimens	0.0806								1641	1600			1,950,000	1,935,000		
T-2		0.0816								1560				1,920,000			

Note: Average coefficient of thermal expansion (75 to 150 F) = $13.2 \times 10^{-6}/^{o}F$.

Table 7

Engineering Properties of Selected Polyester Resin Concrete I, Continuously Graded Quartz-Chert Aggregate, 10% Polyester Resin (7-day Tests)

Specimen No.	Specimen Type	Unit Weight lb/cu in. Single	Unit Weight lb/cu in. Average	Compressive Strength, psi Single	Compressive Strength, psi Average	Modulus of Rupture, psi Single	Modulus of Rupture, psi Average	Tensile Splitting Strength, psi Single	Tensile Splitting Strength, psi Average	Direct Tensile Strength, psi Single	Direct Tensile Strength, psi Average	Secant Modulus Compression, psi 0 to 4000 psi Single	Secant Modulus Compression, psi 0 to 4000 psi Average	Secant Modulus Tension, psi 0 to 600 psi Single	Secant Modulus Tension, psi 0 to 600 psi Average	Shrinkage 7 days	Pot Life
C-3	3- by 6-in. cylinders	0.083		5826								1.4×10^6				Approximately [illegible]	Approximately [illegible]
C-4		0.082		6096	6016							1.64×10^6	1.5×10^6				
C-5		0.083		6125													
P-3'	2-in. cubes	0.0821		6289													
P-4'		0.0818	0.0816	6573	6386												
P-5'		0.0810		629[illegible]													
P-3	2- by 2- by 11-1/4-in. prisms	0.0821				1206											
P-4		0.0818				1300	1254										
P-5		0.0810				1256											
S-3	3- by 6-in. cylinders	ND*						1226									
S-4		ND						1106	1162								
S-5		ND						1153									
T-5	Necked specimens	ND								627	630			2.0×10^6	2.07×10^6		
T-6		ND								633				2.14×10^6			

Note: Average coefficient of thermal expansion (75 to 150 F) = $7.5 \times 10^{-6}/^{o}F$.
* ND = not determined.

Table 8

Engineering Properties of Selected Polyester Resin Concrete II, Continuously Graded Limestone Aggregate, 12% Polyester Resin (7-day Tests)

Specimen		Unit Weight lb/cu in.		Compressive Strength, psi		Modulus of Rupture, psi		Tensile Splitting Strength, psi		Direct Tensile Strength, psi		Secant Modulus Compression, psi 0 to 5000 psi		Secant Modulus Tension, psi 0 to 1000 psi		% Shrinkage 7 days	Pot Life
No.	Type	Single	Average	Single	Average	Single	Average	Single	Average	Single	Average	Single	Average	Single	Average		
C-6 C-7 C-8	3- by 6-in. cylinders	0.0840 0.0857 0.0861		13,333 13,558 13,542	13,478							3.5×10^6 3.6×10^6 3.57×10^6	3.56×10^6			Approximately 0.5	Approximately 15 to 30 min
P-6' P-7' P-8'	2-in. cubes	0.0843 0.0849 0.0864		13,199 13,995 14,567	13,920												
P-6 P-7 P-8	2- by 2- by 11-1/4-in. prisms	0.0843 0.0849 0.0864	0.085			2706 2644 2625	2658										
S-6 S-7	3- by 6-in. cylinders	ND* ND						2129 2176	2152								
T-5 T-6	Necked specimens	ND ND								1443 1133	1288			3,448,000 3,225,000	3,334,000		

Note: Average coefficient of thermal expansion (75 to 150 F) = $6.6 \times 10^{-6}/^{\circ}F$.
* ND = not determined.

Table 4

Strength Increase with Time for Resin Concretes

Type of Resin Concrete	Age at Testing days	Unit Weight lb/cu in. Single	Unit Weight lb/cu in. Average	Modulus of Rupture, psi Single	Modulus of Rupture, psi Average	Compressive Strength, psi Single	Compressive Strength, psi Single	Compressive Strength, psi Average	Flexural Elastic Modulus (Between 600- and 1800-lb Load), psi Single	Flexural Elastic Modulus (Between 600- and 1800-lb Load), psi Average	Note
Epoxy resin concrete (gap-graded quartz-chert aggregate, 16% epoxy resin)	1	0.0796 [illegible]	0.0803	2610 2830	2720	9,200 10,000	9,350 9,920	9,617	887,000 996,000	942,500	2- by 2- by 11-1/4-in. prisms and 2-in. cubes
	3	0.0802 0.0802	0.0802	2560 2660	2610	9,850 9,720	9,150 8,850	9,390	1,450,000 1,450,000	1,450,000	
	7	0.0813 0.0811	0.0812	2760 3000	2880	9,750 10,280	9,400 9,920	9,840	1,450,000 1,330,000	1,390,000	
	14	0.0806 0.0813	0.0809	2870 2850	2860	9,950 9,770	10,480 10,400	10,150	1,450,000 1,330,000	1,390,000	
	28	0.0797 0.0795	0.0796	2610 2720	2660	9,020 8,520	8,590 8,360	8,620	841,000 998,000	919,500	
Polyester resin concrete (continuously graded quartz-chert aggregate, 10% polyester resin)	1	0.0825 0.0829 0.0822	0.0825			4,650 5,250 5,160		5,020			3- by 6-in. cylinders
	3	0.0832 0.0820 0.0820	0.0824			5,390 4,910 5,250		5,180			
	7	0.0831 0.0817 0.0825	0.0824			5,826 6,096 6,125		6,016			
	14	0.0828 0.0833 0.0835	0.0832			5,303 5,981 5,538		5,607			
	21	0.0816 0.0820 0.0830	0.0822			6,170 6,125 6,032		6,109			
	28	0.0814 0.0816 0.0817	0.0816			5,799 6,182 6,624		6,202			
Polyester resin concrete (continuously graded limestone aggregate, 12% polyester resin)	1	0.0864 0.0844 0.0844	0.0851			10,758 9,465 8,779		9,667			3- by 6-in. cylinders
	3	0.0838 0.0863 0.0847	0.0849			11,857 12,849 12,577		12,428			
	7	0.0840 0.0857 0.0861	0.0853			13,332 13,558 13,542		13,476			
	14	0.0880 0.0880 0.0873	0.0878			13,095 12,574 13,106		12,925			
	21	0.0842 0.0853 0.0858	0.0851			12,475 12,762 13,120		12,786			
	28	0.0864 0.0852 0.0842	0.0853			13,142 12,286 12,691		12,706			

Table 10

Summary of Test Results of Epoxy Resin Concrete Beam Series, Series A

Beam No.	Brief Description of Cross Section	Total Ultimate Load, lb: Measured	Total Ultimate Load, lb: Calculated	Measured Cracking Load* lb	Midspan Deflection, in., at a Load of: 3000 lb	6000 lb	9000 lb	12,000 lb	Calculated Cracking Load lb
1A	Two No. 4 reinforcing bars and stirrups (reference beam)	13,000	12,900	(4,000)	0.036	0.100	0.166	0.239	--
2A	Two No. 4 reinforcing bars, stirrups, and 1-1/2-in.-thick layer of epoxy concrete	15,300	12,780	15,000 (6,500)	0.032	0.078	0.137	0.200	11,29[illegible]
3A	One No. 4 reinforcing bar, stirrups, and 1-1/2-in. layer of epoxy concrete	8,100	8,320	3,100 (3,000)	0.058	0.149	--	--	8,320
4A	Stirrups, but no longitudinal reinforcement, 1-1/2-in. layer of epoxy concrete	6,140	5,400	6,140 (6,140)	0.031	0.065	--	--	5,400
5A	Two 1/2-in.-diameter fiber-glass rods, stirrups, and 1-1/2-in.-thick layer of epoxy concrete	16,600	16,260	6,000 (6,000)	0.031	0.202	0.591	0.881	6,080
6A	Two No. 4 reinforcing bars, stirrups, and 3-in.-thick layer of epoxy concrete	14,500	12,980	12,000 (8,500)	0.037	0.072	0.122	0.193	12,980
7A	One No. 4 reinforcing bar, stirrups, and 3-in.-thick layer of epoxy concrete	9,000	9,240	8,000 (5,500)	0.039	0.095	0.193	--	9,240
8A	Stirrups, but no longitudinal reinforcement, 3-in. layer of epoxy concrete	5,500	7,480	5,500 (4,000)	0.068	--	--	--	7,480
9A	No reinforcement, 1-1/2-in. layer of epoxy concrete	3,300	5,140	3,300 (3,000)	0.109	--	--	--	5,140
11A	No reinforcement, 3-in. layer of epoxy concrete	4,200	8,040	4,200 (2,500)	0.187	--	--	--	8,040

* Values in parentheses are the measured cracking loads for the portland cement concrete layer; other values are the measured cracking loads for the epoxy concrete layer.

Table 11

Beam Deflections, Series A

Time, min	Load, lb	Deflections (Midspan), in. 1	2	3	4	5	Remarks
				Beam 1A			
0	0	0	0	0	0	0	
[illegible]	500	0.006	0.007	0.008	0.007	0.004	
10	1,000	0.010	0.012	0.012	0.011	0.007	
[illegible]	1,500	0.014	0.016	0.017	0.015	0.010	
[illegible]	2,000	0.018	0.020	0.021	0.019	0.014	
18		0.016	0.020	0.022	0.020	0.013	
19	2,500	0.020	0.025	0.027	0.025	0.017	
20	3,000	0.024	[illegible]	0.036	0.032	0.021	
24		0.025	[illegible]	0.037	0.033	0.022	
25	1,500	[illegible]	0.026	0.028	0.025	0.017	
27	0	0.004	0.006	0.007	0.006	0.004	
29		0.004	0.006	0.006	0.006	0.004	
30	1,500	0.017	0.022	0.024	0.022	0.014	
31	3,000	0.026	0.035	0.038	0.036	0.022	
34		0.026	0.036	0.039	0.035	0.023	
35	3,500	0.030	0.043	0.046	0.041	0.026	
36	4,000	0.036	0.052	0.059	0.052	0.033	
39		0.041	0.060	0.066	0.060	0.038	First hairline cracks in concrete observed
41	4,500	0.042	0.063	0.071	0.063	0.040	
43	5,000	0.047	0.069	0.079	0.069	0.044	
45		0.047	0.071	0.081	0.071	0.045	
46	5,500	0.052	0.078	0.089	0.078	0.049	
47	6,000	0.057	0.087	0.099	0.087	0.055	
55		0.058	0.088	0.100	0.088	0.055	
56	3,000	0.044	0.065	0.073	[illegible]	0.041	
60	0	0.012	0.018	0.020	0.017	0.014	
62	0	0.012	0.017	0.019	0.017	0.014	
63	3,000	0.038	0.056	0.063	0.055	0.037	
64	6,000	0.059	0.091	0.104	0.091	0.059	
65		0.060	0.092	0.104	0.091	0.059	
68	6,500	0.063	0.098	0.111	0.098	0.063	
69	7,000	0.068	0.105	0.120	0.105	0.068	
71		0.069	0.107	0.121	0.107	0.069	
74	7,500	0.075	0.116	0.131	0.116	0.076	
75	8,000	0.080	0.124	0.140	0.124	0.081	
78	8,500	0.086	0.135	0.153	0.134	0.089	
80	9,000	0.093	0.145	0.164	0.144	0.096	
81		0.094	0.147	0.166	0.146	0.098	
83	6,000	0.070	0.121	0.136	0.120	0.082	
84	3,000	0.055	0.082	0.090	0.079	0.058	
85	0	0.018	0.027	0.029	0.027	0.024	
87		0.017	0.026	0.028	0.026	0.024	
[illegible]	3,000	[illegible]	[illegible]	[illegible]	0.067	0.050	
89	6,000	0.072	0.110	0.123	0.109	0.076	
90	9,000	0.096	0.150	0.169	0.148	0.101	
95		0.098	0.152	0.171	0.150	0.103	
97	9,500	0.102	0.160	0.180	0.158	0.107	
98	10,000	0.107	0.168	0.189	0.166	0.113	
101		0.109	0.171	0.193	0.170	0.116	
102	10,500	0.113	0.178	0.200	0.177	0.120	
103	11,000	0.120	0.188	0.211	0.188	0.127	
106	11,500	0.127	0.199	0.223	0.199	0.135	
110	12,000	0.132	0.208	0.233	0.209	0.141	
114		0.136	0.213	0.239	0.214	0.144	
116	9,000	0.122	0.191	0.212	0.190	0.130	
118	6,000	0.100	0.152	0.168	0.151	0.107	
119	3,000	0.068	0.102	0.111	0.102	0.077	
120	0	0.027	0.043	0.044	0.043	0.038	
123		0.026	0.042	0.043	0.042	0.037	
125	3,000	0.057	0.085	0.093	0.085	0.064	
126	6,000	0.085	0.132	0.146	0.131	0.094	
127	9,000	0.113	0.177	0.198	0.176	0.121	
128	12,000	0.139	0.219	0.244	0.219	0.147	
132		0.146	0.232	0.258	0.230	0.155	
133	12,500	0.160	0.260	0.288	0.255	0.169	
136	13,000	0.240	0.430	0.500	0.420	0.280	
140		0.288	0.507	0.596	0.480	0.317	
155	13,000			0.720			Compressional concrete failure
				Beam 2A			
0	0	0	0	0	0	0	
19	500	0.002	0.008	0.008	0.007	0.003	
20	1,000	0.010	0.012	0.013	0.011	0.007	
22		0.011	0.013	0.014	0.012	0.007	
24	1,500	0.014	0.017	0.018	0.015	0.009	
25	2,000	0.017	0.021	0.022	0.019	0.011	
26		0.017	0.021	0.022	0.019	0.012	
27	2,500	0.020	0.025	0.027	0.023	0.014	
28	3,000	0.022	0.029	0.031	0.028	0.017	
29		0.023	0.030	0.032	0.029	0.017	
				Beam 2A (Continued)			
30	1,500	0.017	0.022	0.023	[illegible]	0.012	
31	0	0.003	0.003	0.003	[illegible]	0.005	
33		0.002	0.002	0.003	[illegible]	0.005	
34	1,500	0.015	0.019	0.020	[illegible]	0.011	
35	3,000	0.023	0.029	0.032	0.028	0.017	
37		0.024	0.031	0.033	0.029	0.018	
38	3,500	0.027	0.035	0.038	0.033	0.021	
39	4,000	0.029	0.040	0.044	0.038	0.024	
41		0.030	0.042	0.045	0.039	0.025	
42	4,500	0.033	0.046	0.051	0.044	0.028	
43	5,000	0.037	0.053	0.058	0.050	0.032	
45		0.038	0.055	0.061	0.052	0.033	
46	5,500	0.041	0.060	0.067	0.057	0.037	
47	6,000	0.044	0.066	0.074	0.063	0.040	
48		0.046	0.069	0.078	0.066	0.041	
49	3,000	0.034	0.049	0.055	0.046	0.029	
50	0	0.008	0.010	0.014	0.011	0.007	
52	0	0.007	0.008	0.012	0.009	[illegible]	
53	3,000	0.028	0.040	0.045	0.037	0.025	
54	6,000	0.045	0.067	0.075	0.064	0.040	
55		0.047	0.070	0.080	0.068	0.043	
56	6,500	0.051	0.076	0.086	0.074	0.046	First hairline cracks in portland cement concrete
60	7,000	0.056	0.085	0.096	0.083	0.051	
63		0.057	0.087	0.099	0.085	0.053	
65	7,500	0.060	0.092	0.105	0.091	0.056	
66	8,000	0.064	0.098	0.112	0.098	0.060	
68		0.066	0.101	0.116	0.101	0.062	
70	8,500	0.070	0.108	0.124	0.107	0.066	
71	9,000	0.074	0.115	0.132	0.115	0.070	
75		0.077	0.119	0.137	0.119	0.072	
76	6,000	0.065	0.099	0.114	0.099	0.060	
77	3,000	0.047	0.069	0.081	0.069	0.042	
78	0	0.016	0.022	0.030	0.025	0.016	
80		0.012	0.016	0.024	0.019	0.011	
81	3,000	0.036	0.051	0.060	0.050	0.030	
82	6,000	0.054	0.081	0.092	0.079	0.049	
83	9,000	0.074	0.114	0.132	0.113	0.068	
84		0.077	0.119	0.138	0.118	0.070	
85	9,500	0.081	0.125	0.145	0.124	0.074	
86	10,000	0.085	0.132	0.152	0.131	0.077	
90		0.088	0.138	0.159	0.137	0.080	
91	10,500	0.092	0.143	0.165	0.142	0.083	
[illegible]	11,000	0.096	0.150	0.173	0.148	0.086	
96		0.099	0.156	0.180	0.155	0.090	
97	11,500	0.103	0.162	0.186	0.160	0.092	
98	12,000	0.107	0.168	0.194	0.167	0.096	
100		0.110	0.174	0.200	0.173	0.099	
102	9,000	0.099	0.155	0.180	0.154	0.088	
103	6,000	0.081	0.122	0.143	0.122	0.071	
104	3,000	0.059	0.086	0.101	0.086	0.052	
105	0	0.024	0.032	0.042	0.035	0.033	
107		0.020	0.026	0.036	0.029	0.033	
108	3,000	0.043	0.063	0.073	0.061	0.038	
109	6,000	0.064	0.097	0.110	0.096	0.058	
110	9,000	0.084	0.131	0.151	0.130	0.078	
111	12,000	0.106	0.167	0.193	0.166	0.096	
113		0.112	0.177	0.204	0.176	0.101	
114	12,500	0.117	0.184	0.212	0.183	0.104	
115	13,000	0.121	0.191	0.220	0.190	0.107	
118		0.125	0.198	0.227	0.197	0.111	
119	13,500	0.129	0.204	0.235	0.204	0.115	
120	14,000	0.134	0.212	0.244	0.211	0.119	
123		0.139	0.221	0.256	0.220	0.123	
124	14,500	0.146	0.232	0.270	0.231	0.129	
126	15,000	0.159	0.257	0.300	0.255	0.142	
130	15,000	0.250	0.440	0.550	0.330	0.230	
133	15,000			0.610			Crack in epoxy concrete
135	15,300			0.800			Compressional concrete failure
137	0			0.590			
				Beam 3A			
0	0	0	0	0	0	0	
3	500	0.008	0.010	0.010	0.008	0.006	
4	1,000	0.011	0.015	0.015	0.014	0.010	
5		0.011	0.015	0.015	0.014	0.010	
6	1,500	0.015	0.021	0.022	0.020	0.014	
7	2,000	0.019	0.026	0.027	0.025	0.018	
8		0.020	0.027	0.029	0.026	0.018	
9	2,500	0.023	0.033	0.035	0.032	0.022	
10	3,000	0.031	0.047	0.052	0.045	0.030	
14		0.034	0.053	0.058	0.050	0.032	

(Continued)

Time min	Load lb	Deflections (Midspan), in. 1	2	3	4	5	Remarks
				Beam 3A (Continued)			
15	1,500	0.027	0.044	0.044	0.037	[illegible]	
16	0	0.006	0.009	0.010	0.009	0.006	
17		0.005	0.008	0.009	0.009	0.005	
18	1,500	0.021	0.031	0.033	0.030	0.020	
19	3,000	0.033	0.031	0.056	0.049	[illegible]	
20		0.039	0.054	0.060	0.051	0.033	First hairline cracks in portland cement concrete
21	3,500	0.039	0.063	0.069	0.059	0.038	
22	4,000	0.045	0.073	0.080	0.069	0.043	
23		0.049	0.080	0.088	0.075	0.047	
24	4,500	0.054	0.088	0.097	0.083	0.052	
25	5,000	0.060	0.099	0.111	0.097	0.060	
28		0.064	0.105	0.118	0.103	0.063	
30	5,500	0.070	0.115	0.129	0.113	0.068	
31	6,000	0.076	0.126	0.141	0.124	0.075	
34		0.080	0.134	0.149	0.131	0.079	
35	3,000	0.060	0.059	0.107	0.094	0.057	
36	0	0.015	0.026	0.027	0.023	0.014	
38	0	0.011	0.020	0.021	0.018	0.011	
39	3,000	0.046	0.075	0.082	0.071	0.044	
40	6,000	0.079	0.130	0.145	0.127	0.077	
42		0.083	0.139	0.154	0.135	0.081	
43	6,500	0.089	0.148	0.165	0.145	0.086	
44	7,000	0.095	0.160	0.178	0.156	0.093	
46		0.098	0.166	0.184	0.162	0.096	
47	7,500	0.107	0.182	0.203	0.178	0.105	
48	8,000	0.146	0.218	0.249	0.215	0.127	
50	8,000		0.270	0.320	0.270		
52	8,000			0.300			
53	8,100			1.000			Crack in epoxy concrete
55	0			0.650			
				Beam 4A			
0	0	0	0	0	0	0	
3	500	0.009	0.009	0.008	0.007	0.005	
4	1,000	0.012	0.013	0.013	0.011	0.008	
6		0.012	0.013	0.013	0.011	0.008	
7	1,500	0.015	0.018	0.017	0.015	0.010	
9	2,000	0.018	0.022	0.022	0.019	0.013	
10		0.018	0.022	0.022	0.019	0.013	
11	2,500	0.021	0.026	0.026	0.022	0.015	
13	3,000	0.024	0.030	0.031	0.027	0.018	
15		0.024	0.030	0.031	0.027	0.018	
16	1,500	0.018	0.022	0.021	0.019	0.013	
17	0	0.003	0.002	0.002	0.002	0.003	
19		0.002	0.002	0.001	0.002	0.003	
20	1,500	0.017	0.020	0.020	0.017	0.012	
21	3,000	0.024	0.031	0.031	0.027	0.018	
23		0.024	0.031	0.032	0.028	0.018	
24	3,500	0.027	0.035	0.036	0.032	0.021	
26	4,000	0.029	0.039	0.040	0.035	0.023	
28		0.029	0.039	0.040	0.035	0.023	
30	4,500	0.032	0.043	0.045	0.040	0.025	
32	5,000	0.035	0.048	0.051	0.044	0.028	
34		0.035	0.048	0.051	0.044	0.028	
35	5,500	0.038	0.053	0.057	0.049	0.031	
37	6,000	0.042	0.059	0.063	0.055	0.035	
40		0.042	0.061	0.065	0.057	0.035	
41	3,000	0.031	0.042	0.044	0.037	0.024	
42	0	0.005	0.006	0.006	0.006	0.004	
45	0	0.004	0.005	0.005	0.005	0.004	
46	3,000	0.029	0.038	0.040	0.035	0.022	
47	6,000	0.043	0.061	0.066	0.057	0.036	
50		0.045	0.064	0.069	0.060	0.037	
61	6,140			Sudden failure			
				Beam 5A			
0	0	0	0	0	0	0	
3	500	0.005	0.007	0.007	0.005	0.004	
4	1,000	0.009	0.011	0.011	0.009	0.006	
6		0.009	0.011	0.012	0.009	0.006	
7	1,500	0.012	0.015	0.016	0.013	0.009	
8	2,000	0.015	0.019	0.020	0.017	0.011	
10		0.015	0.019	0.020	0.017	0.011	
11	2,500	0.018	0.023	0.025	0.021	0.014	
12	3,000	0.020	0.027	0.030	0.025	0.017	
15		0.021	0.027	0.031	0.026	0.017	
17	1,500	0.015	0.018	0.021	0.017	0.012	
18	0	0.002	0.003	0.003	0.003	0.002	
21		0.002	0.002	0.003	0.003	0.002	
22	1,500	0.013	0.016	0.017	0.015	0.010	
23	3,000	0.020	0.028	0.030	0.025	0.017	
25		0.021	0.028	0.030	0.026	0.017	
26	3,500	0.023	0.030	0.035	0.029	0.019	
27	4,000	0.026	0.034	0.039	0.033	0.022	
				Beam 5A (Continued)			
28		0.027	0.035	0.041	0.035	0.023	
29	4,500	0.029	0.038	0.044	0.038	[illegible]	
30	5,000	0.032	0.043	0.050	0.043	[illegible]	
31		0.033	0.045	0.052	0.044	0.029	
34	5,500	0.035	0.048	0.055	0.048	0.031	
37	6,000	0.060	0.098	0.128	0.100	0.061	First cracks in portland cement and epoxy concrete
38		0.078	0.147	0.165	0.160	0.090	
44		0.108	0.189	0.204	0.189	0.104	
46	3,000	0.075	0.136	0.144	0.132	0.075	
48	0	0.023	0.043	0.044	0.046	0.025	
51		0.023	0.042	0.042	0.045	0.025	
53	3,000	0.059	0.109	0.115	0.108	0.062	
55	6,000	0.104	0.194	0.204	0.186	0.105	
58		0.134	0.251	0.278	0.242	0.134	
65		0.141	0.264	0.293	0.255	0.141	
67	6,500	0.151	0.284	0.315	0.275	0.154	
70	7,000	0.166	0.312	0.350	0.302	0.170	
76		0.173	0.323	0.366	0.320	0.180	
78	7,500	0.186	0.347	0.398	0.346	0.196	
[illegible]	8,000	0.199	0.370	0.424	0.372	0.210	
85		0.217	0.404	0.471	0.420	0.231	
86	8,500	0.230	0.429	0.502	0.448	0.246	
89	9,000	0.272	0.506	0.578	0.511	0.283	
95	9,000	0.279	0.518	0.591	0.525	0.289	
96	6,000	0.225	0.417	0.479	0.425	0.225	
97	3,000	0.160	0.292	0.337	0.298	0.166	
101	0	0.062	0.111	0.129	0.114	0.068	
104		0.061	0.109	0.126	0.112	0.066	
105	3,000	0.114	0.211	0.238	0.214	0.122	
106	6,000	0.198	0.366	0.415	0.368	0.205	
108	9,000	0.280	0.519	0.595	0.530	0.292	
111	9,500	0.299	0.555	0.638	0.572	0.312	
113	10,000	0.340	0.615	0.700	0.629	0.336	
119		--	0.630	0.711	0.640	0.344	
121	10,500		0.656	0.742	0.670	0.363	
122	11,000		0.686	0.775	0.702	0.383	
124	11,500		0.733	0.824	0.752	0.408	
126	12,000		0.767	0.867	0.792	0.429	
129			0.782	0.881	0.805	0.425	
131	9,000		0.715	0.804	0.738	0.399	
132	6,000		0.562	0.635	0.587	0.320	
133	3,000		0.369	0.419	0.382	0.204	
135	0		0.150	0.171	0.160	0.092	
185			0.136	0.153	0.147	0.085	
188	3,000		0.255	0.290	0.265	0.150	
189	6,000		0.434	0.497	0.446	0.246	
190	9,000		0.612	0.694	0.630	0.340	
195	12,000		0.820	0.924	0.848	0.460	
198				0.929			
198				0.930			
200				0.980			
201				1.010			
203	13,000			1.050			
205	13,500			1.080			
206	14,000			1.140			
208	14,500			1.180			
209	15,000			1.260			
210	15,500			1.310			
211	16,000			1.390			
212	16,600			1.580			Compressional concrete failure
220	0			1.480			
				Beam 6A			
0	0	0	0	0	0	0	
5	500	0.005	0.006	0.007	0.006	0.004	
6	1,000	0.010	0.012	0.013	0.011	0.004	
8		0.010	0.013	0.015	0.012	0.009	
9	1,500	0.013	0.017	0.019	0.016	0.011	
10	2,000	0.016	0.022	0.024	0.020	0.014	
11		0.017	0.023	0.026	0.022	0.015	
12	2,500	0.020	0.027	0.031	0.026	0.017	
13	3,000	0.022	0.031	0.035	0.030	0.020	
14		0.025	0.033	0.037	0.032	0.021	
15	1,500	0.016	0.022	0.025	0.021	0.024	
16	0	0.004	0.004	0.005	0.004	0.004	
18		0.003	0.004	0.004	0.003	0.004	
19	1,500	0.014	0.019	0.021	0.018	0.012	
20	3,000	0.023	0.032	0.036	0.031	0.025	
23		0.023	0.033	0.037	0.032	0.021	
24	3,500	0.026	0.037	0.042	0.036	0.024	
25	4,000	0.029	0.042	0.047	0.041	0.027	
27		0.030	0.043	0.049	0.042	0.027	
29	4,500	0.033	0.048	0.054	0.046	0.030	

(Continued)

Load No.	Load, lb	Deflection, in.					Remarks
		Beam [illegible] (Continued)					
[illegible]	[illegible]	0.036	[illegible]	0.059	[illegible]	[illegible]	
[illegible]		0.036	0.053	0.060	0.050	0.033	
[illegible]	[illegible]	0.039	0.058	0.065	0.054	0.036	
[illegible]	[illegible]	0.042	0.061	0.071	0.061	0.039	
[illegible]		0.042	0.064	0.072	0.062	0.040	
36	3,000	0.030	0.045	0.051	0.043	0.028	
38	0	0.008	0.012	0.012	0.010	0.005	
39	3,000	0.026	0.038	0.041	0.037	0.024	
41	6,000	0.049	0.064	0.072	0.061	0.039	
43		0.041	0.066	0.074	0.061	0.040	
44	6,500	0.046	0.071	0.079	0.066	0.043	
45	7,000	0.048	0.075	0.084	0.072	0.046	
47		0.049	0.077	0.086	0.075	0.047	
49	7,500	0.052	0.082	0.093	0.080	0.051	
50	8,000	0.056	0.088	0.099	0.086	0.054	
53		0.057	0.092	0.103	0.089	0.055	
55	8,500	0.060	0.098	0.109	0.094	0.058	First hairline cracks in portland cement concrete
57	9,000	0.064	0.104	0.117	0.102	0.062	
61		0.066	0.109	0.122	0.106	0.065	
63	6,000	0.056	0.092	0.103	0.089	0.055	
64	3,000	0.040	0.064	0.071	0.060	0.038	
66	0	0.014	0.022	0.025	0.021	0.012	
68		0.012	0.018	0.022	0.018	0.010	
70	3,000	0.033	0.051	0.057	0.048	0.030	
71	6,000	0.049	0.079	0.088	0.076	0.046	
72	9,000	0.066	0.106	0.120	0.103	0.064	
75		0.070	0.114	0.129	0.112	0.069	
77	9,500	0.072	0.118	[illegible]	0.116	0.072	
79	10,000	0.075	0.122	0.138	0.120	0.074	
83		0.078	0.129	0.146	0.127	0.078	
84	10,500	0.081	0.133	0.152	0.133	0.082	
85	11,000	0.085	0.140	0.160	0.140	0.086	
90		0.089	0.148	0.169	0.149	0.091	
91	11,500	0.092	0.153	0.174	0.154	0.044	
93	12,000	0.097	0.161	0.183	0.162	0.099	
98		0.100	0.169	0.193	0.172	0.104	
99	9,000	0.092	0.153	0.166	0.154	0.094	
100	6,000	0.075	0.123	0.140	0.125	0.077	
101	3,000	0.053	0.089	0.096	0.086	0.062	
102	0	[illegible]	0.029	0.044	0.032	[illegible]	
105		0.019	0.032	0.037	0.033	0.020	
107	3,000	0.043	0.068	0.077	0.068	0.042	
108	6,000	0.061	0.100	0.114	0.100	0.061	
109	9,000	0.080	0.134	0.153	0.134	0.082	
110	12,000	0.100	0.165	0.189	0.169	0.102	Crack in epoxy concrete
112		0.105	0.177	0.201	0.179	0.108	
113	12,500	0.110	0.183	0.209	0.186	0.112	
114	13,000	0.113	0.190	0.216	0.192	0.116	
117		0.118	0.199	0.228	0.203	0.118	
118	13,500	0.122	0.206	0.236	0.211	0.126	
119	14,000	0.127	0.213	0.247	0.220	0.131	
[illegible]		0.188	0.330	0.420	0.393	0.220	
125		0.192	0.343	0.441	0.415	0.231	
127	14,500	0.215	0.374	0.485	0.470	0.158	
128		0.228	0.405	0.530	0.517	0.182	
129			0.440	0.700	0.500		Compressional concrete failure
134	0			0.485			
		Beam 7A					
0	0	0	0	0	0	0	
8	500	0.008	0.008	0.009	0.009	0.010	
10	1,000	0.012	0.014	0.013	0.013	0.011	
13		0.013	0.013	0.015	0.012	0.012	
15	1,500	0.015	0.017	0.020	0.016	0.014	
16	2,000	0.019	0.021	0.025	0.021	0.018	
18		0.019	0.022	0.026	0.021	0.018	
19	2,500	0.022	0.026	0.031	0.025	0.020	
20	3,000	0.026	0.031	0.037	0.030	0.023	
23		0.027	0.033	0.039	0.031	0.024	
25	1,500	0.020	0.022	0.026	0.022	0.018	
26	0	0.005	0.005	0.005	0.004	0.008	
28		0.004	0.003	0.004	0.003	0.008	
29	1,500	0.017	0.019	0.022	0.018	0.015	
30	3,000	0.027	0.033	0.038	0.032	0.024	
31		0.027	0.033	0.039	0.032	0.024	
32	3,500	0.031	0.038	0.045	0.037	0.027	
33	4,000	0.034	0.042	0.050	0.041	0.030	
34		0.035	0.044	0.052	0.042	0.031	
35	4,500	0.038	0.049	0.057	0.047	0.033	
36	5,000	0.042	0.057	0.065	0.053	0.036	
37		0.044	0.060	0.068	0.055	0.038	
38	5,500	0.049	0.068	0.076	0.061	0.041	Hairline cracks in portland cement concrete
40	6,000	0.056	0.079	0.086	0.070	0.045	
49		0.061	0.089	0.095	0.075	0.049	
		Beam 7A (Continued)					
[illegible]	3,000	0.048	[illegible]	0.071	0.056	0.038	
[illegible]	0	0.014	0.018	0.019	0.013	0.010	
[illegible]		0.013	0.016	0.016	0.018	0.010	
[illegible]	3,000	0.038	[illegible]	[illegible]	[illegible]	0.031	
[illegible]	6,000	[illegible]	0.084	0.091	0.073	0.048	
[illegible]		0.063	0.092	0.099	0.078	0.051	
[illegible]	6,500	0.066	0.098	0.103	0.082	0.053	
[illegible]	7,000	0.070	0.104	0.110	0.088	0.056	
[illegible]		0.071	0.109	0.116	0.091	0.058	
[illegible]	7,500	0.079	0.120	0.130	0.100	0.064	
[illegible]	8,000	0.084	0.129	0.141	0.108	0.069	Crack in epoxy concrete
[illegible]		0.092	0.144	0.159	0.120	0.075	
[illegible]	8,500	0.096	0.152	0.167	0.125	0.079	
[illegible]	9,000	0.106	0.170	0.193	0.145	0.070	Failure
[illegible]	7,000			0.275			
86	7,400			0.400			
88	0			0.275			
		Beam 8A					
0	0	0	0	0	0	0	
3	500	0.005	0.008	0.008	0.007	0.004	
4	1,000	0.010	0.012	0.016	0.014	0.007	
7		0.013	0.018	0.021	0.019	0.011	
8	1,500	0.017	0.024	0.028	0.025	0.015	
9	2,000	0.022	0.033	0.037	0.032	0.021	
10		0.023	0.034	0.039	0.034	0.021	
11	2,500	0.028	0.042	0.048	0.043	0.026	
12	3,000	0.033	0.051	0.058	0.052	0.032	
14		0.036	0.058	0.068	0.060	0.036	
13	1,500	0.025	0.046	0.053	0.047	0.028	
15	0	0.012	0.015	0.021	0.020	0.011	
16		0.008	0.015	0.016	0.015	0.006	
19	1,500	0.022	0.033	0.038	0.035	0.021	
20	3,000	0.035	0.056	0.064	0.057	0.035	
22		0.040	0.065	0.075	0.067	0.041	
23	3,500	0.045	0.073	0.083	0.075	0.047	
24	4,000	0.050	0.082	0.094	0.085	0.053	Hairline cracks in portland cement concrete
26		0.059	0.099	0.114	0.103	0.065	
27	4,500	0.067	0.112	0.130	0.118	0.074	
29	5,000	[illegible]	0.132	0.155	0.140	0.087	
31		0.095	0.165	0.194	0.174	0.108	
32	5,500	Sudden failure					
		Beam 9A					
0	0	0	0	0	0	0	
2	500	0.004	0.007	0.007	0.006	0.004	
3	1,000	0.008	0.013	0.013	0.012	0.008	
5		0.010	0.015	0.015	0.013	0.009	
6	1,500	0.013	0.021	0.020	0.018	0.012	
7	2,000	0.016	0.026	0.026	0.023	0.015	
8		0.017	0.027	0.027	0.024	0.016	
9	2,500	0.022	0.036	0.036	0.032	0.021	
10	3,000	0.031	0.058	0.067	0.060	0.037	
13		0.046	0.092	0.109	0.100	0.057	
15	1,500	0.041	0.084	0.099	0.089	0.050	
16	0	0.021	0.047	[illegible]	0.040	0.027	
18		0.018	0.041	0.044	0.040	0.022	
20	1,500	0.031	0.064	0.074	0.066	0.038	
21	3,000	0.049	0.099	0.111	0.107	0.061	Hairline cracks in portland cement concrete
24		0.073	0.135	0.168	0.156	0.087	
26	3,300	Sudden failure					
		Beam 11A					
0	0	0	0	0	0	0	
3	500	0.011	0.015	0.016	0.014	0.009	
4	1,000	0.013	0.019	0.021	0.019	0.012	
5		0.014	0.020	0.022	0.020	0.012	
6	1,500	0.020	0.030	0.032	0.029	0.018	
7	2,000	0.026	0.041	0.044	0.040	0.025	
8		0.033	0.052	0.057	0.051	0.031	
9	2,500	0.041	0.068	0.076	0.068	0.041	Hairline cracks in portland cement concrete
12	3,000	0.068	0.118	0.139	0.119	0.070	
15		0.089	0.158	0.187	0.158	0.091	
17	1,500	0.081	0.144	0.171	0.143	0.082	
19	0	0.040	0.075	0.087	0.073	0.040	
21		0.031	0.057	0.065	0.056	0.031	
26	1,500	0.050	0.074	0.085	0.075	0.044	
29	3,000	0.074	0.130	0.155	0.133	0.079	
32		0.100	0.180	0.210	0.178	0.101	
33	3,500	0.113	0.210	0.249	0.210	0.120	
35		0.138	0.251	0.294	0.249	0.142	
37	4,000	0.155	0.283	0.330	0.280	0.159	
40		0.210	0.394	0.460	0.380	0.209	
42	4,800	Sudden failure					

Table 12

Moment-Curvature Relations, Series A

Moment ft-lb	$\frac{\epsilon_r - \epsilon_c}{d}$ 10^{-6} Radians	$\frac{\epsilon_{8,9,12,13} - \epsilon_c}{a_1}$ 10^{-6} Radians	$\frac{\epsilon_{10,11,14,15} - \epsilon_c}{a_2}$ 10^{-6} Radians	Average ϕ 10^{-6} Radians
Beam 1A				
(d = 8.00 in.; a_1 = 8.00 in.; a_2 = 8.85 in.)				
0	0	0	0	0
1,000	11.2	12.5	11.3	11.7
2,000	22.4	24.6	22.4	23.1
3,000	46.6	43.4	39.2	43.0
0	11.2	9.4	16.8	12.5
3,000	47.2	43.4	50.4	47.0
4,000	91.0	102.4	106.6	100.0
5,000	129.0	142.6	131.8	134.4
6,000	160.7	179.9	162.6	167.7
0	34.9	37.1	36.4	36.1
6,000	166.1	173.5	162.6	167.4
7,000	191.0	192.1	185.0	189.4
8,000	223.0	217.0	212.9	217.6
9,000	252.0	235.6	227.0	238.2
0	40.5	34.1	36.4	37.0
9,000	260.0	241.9	241.0	247.6
10,000	284.0	263.5	263.4	270.3
11,000	315.0	288.3	288.6	297.3
12,000	344.0	316.1	319.4	326.5
0	51.1	49.6	47.7	49.5
12,000	355.0	356.5	336.3	349.3
Beam 2A				
(d = 8.00 in.; a_1 = 8.00 in.; a_2 = 8.85 in.)				
0	0	0	0	0
1,000	10.6	12.4	8.5	10.5
2,000	26.1	27.9	22.4	25.5
3,000	39.9	40.4	35.5	38.9
0	8.8	15.5	11.3	11.8
3,000	36.9	34.1	36.4	35.8
4,000	44.8	49.5	53.2	52.5
5,000	85.4	86.9	84.1	85.5
6,000	112.0	114.8	112.1	113.0
Beam 2A (Continued)				
(d = 8.00 in.; a_1 = 8.00 in.; a_2 = 8.85 in.)				
0	16.2	18.6	14.0	15.3
6,000	115.0	117.8	112.1	114.9
7,000	151.0	158.8	148.5	152.8
8,000	178.0	182.9	173.7	178.2
9,000	206.0	210.9	204.7	207.2
0	31.2	45.5	36.5	38.1
9,000	205.0	220.4	207.3	210.9
10,000	235.0	251.1	232.5	239.5
11,000	265.0	278.9	263.4	269.1
12,000	295.0	307.0	291.5	297.8
0	35.5	58.9	44.9	46.4
12,000	293.0	307.0	291.9	297.3
13,000	330.0	341.0	327.9	333.0
14,000	358.0	387.5	375.6	373.7
15,000	413.0			
Beam 3A				
(d = 8.00 in.; a_1 = 7.91 in.; a_2 = 8.76 in.)				
0	0	0	0	0
1,000	9.4	6.3	8.4	8.0
2,000	25.6	24.9	25.3	25.3
3,000	65.5	77.5	72.9	72.0
0	24.3	18.6	16.8	19.9
3,000	77.4	89.9	78.5	81.9
4,000	128.0	155.0	140.1	141.0
5,000	180.0	241.9	204.6	208.8
6,000	227.0	322.5	277.5	275.7
0	10.6	55.9	33.7	33.4
6,000	240.0	316.3	280.2	278.8
7,000	284.0	365.9	330.7	326.8
8,000	326.0	778.0	683.7	595.9

(Continued)

Table 12 (Concluded)

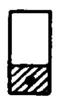
Moment ft-lb	$\frac{\epsilon_r - \epsilon_c}{d}$ 10^{-6} Radians	$\frac{\epsilon_{8,9,12,13} - \epsilon_c}{a_1}$ 10^{-6} Radians	$\frac{\epsilon_{10,11,14,15} - \epsilon_c}{a_2}$ 10^{-6} Radians	Average ϕ 10^{-6} Radians
		Beam 7A		
		(d = 8.00 in.; a_1 = 8.05 in.; a_2 = 8.90 in.)		
0	0	0	0	0
1,000	15.0	15.4	16.7	15.7
2,000	26.1	24.6	19.4	23.4
3,000	38.0	33.9	33.5	35.1
0	3.0	6.2	5.6	4.9
3,000	41.0	37.0	39.0	39.0
4,000	56.0	49.2	50.1	51.8
5,000	72.0	64.7	66.9	67.9
6,000	110.0	92.4	100.3	100.9
0	14.0	12.3	13.9	13.4
6,000	111.0	92.3	100.2	101.2
7,000	138.0	132.5	142.1	137.5
8,000	289.0	234.0	362.2	295.1
		Beam 8A		
		(d = 8.00 in.; a_1 = 7.99 in.; a_2 = 8.34 in.)		
0	NA	0	0	0
1,000	↓	24.8	28.1	26.5
2,000	↓	55.8	58.8	57.3
3,000	↓	93.1	98.2	95.7
0	↓	31.0	36.4	33.7
3,000	↓	143.1	112.2	127.7
4,000	↓	164.5	154.8	159.7
5,000	↓	279.3	297.4	288.4

Moment ft-lb	$\frac{\epsilon_r - \epsilon_c}{d}$ 10^{-6} Radians	$\frac{\epsilon_{8,9,12,13} - \epsilon_c}{a_1}$ 10^{-6} Radians	$\frac{\epsilon_{10,11,14,15} - \epsilon_c}{a_2}$ 10^{-6} Radians	Average ϕ 10^{-6} Radians
		Beam 9A		
		(d = 8.00 in.; a_1 = 8.02 in.; a_2 = 8.87 in.)		
0	NA*	0	0	0
1,000	↓	12.5	11.3	11.9
2,000	↓	37.2	33.6	35.4
3,000	↓	126.7	134.2	130.5
0	↓	86.5	83.9	85.2
[illegible],000	↓	225.7	237.7	231.7
		Beam 11A		
		(d = 8.00 in.; a_1 = 7.99 in.; a_2 = 8.84 in.)		
		0	0	0
1,000	NA	34.0	22.4	28.2
2,000	↓	80.6	78.5	79.6
3,000	↓	282.5	291.7	287.1
0	↓	155.2	145.9	150.6
3,000	↓	334.5	356.3	345.4
4,000	↓	707.6	779.9	743.8

* NA = not applicable.

Table 13

Summary of Test Results of Polyester Resin Concrete Beam Series, Series B

Beam No.		Brief Description of Cross Section	Total Ultimate Load, lb		Measured Cracking Load* lb	Midspan Deflection, in., at a Load of				Calculated Cracking Load lb
			Measured	Calculated		3000 lb	6000 lb	9000 lb	12,000 lb	
1B		Two No. 4 reinforcing bars, stirrups (reference beam)	12,280	12,000	(3,000)	0.058	0.124	0.196	0.283	--
6B		Two No. 4 reinforcing bars, stirrups, 3-in. layer polyester resin concrete	12,000	11,520	9,000 (10,000)	0.050	0.117	0.191	0.291	2950
7B		One No. 4 reinforcing bar, stirrups, 3-in. layer polyester resin concrete	6,840	6,540	6,000 (5,000)	046	0.154	--	--	3620
8B		No reinforcement 3-in. layer polyester resin concrete	3,580	2,950	--	0.070	--	--	--	2950

* Values in parentheses are the measured cracking loads for the layer of portland cement concrete; other values are the measured cracking loads for the polyester concrete layer.

Table 14

Beam Deflections, Series B

Time min	Load lb	Deflections (Midspan), in. 1	2	3	4	5
			Beam 1B			
0	0	0	0	0	0	0
26	1,000	0.012	0.015	0.015	0.012	0.009
30	2,000	0.025	0.032	0.034	0.028	0.019
33	2,300	0.026	0.034	0.036	0.030	0.020
34	3,000	0.040	0.053	0.058	0.048	0.031
40	4,000	0.055	0.074	0.080	0.068	0.043
44	4,000	0.057	0.078	0.084	0.071	0.045
47	5,000	0.072	0.092	0.102	0.087	0.054
50	6,000	0.081	0.112	0.124	0.106	0.065
54	6,000	0.085	0.118	0.130	0.110	0.068
59	0	0.017	0.027	0.030	0.025	0.016
63	0	0.017	0.027	0.029	0.024	0.016
69	7,000	0.095	0.135	0.149	0.120	0.079
71	8,000	0.108	0.155	0.172	0.147	0.089
76	8,000	0.113	0.162	0.179	0.154	0.093
81	9,000	0.122	0.176	0.196	0.160	0.102
82	10,000	0.138	0.200	0.223	0.194	0.120
86	10,000	0.143	0.208	0.231	0.201	0.124
88	11,000	0.154	0.225	0.251	0.219	0.134
92	12,000	0.167	0.264	0.283	0.249	0.156
95	12,000	0.209	.332	0.395	0.327	0.196
100	0	0.098	0.163	0.210	0.158	0.089
105	12,280			Failure		
110	0	0.187	0.332	0.455	0.341	0.175
			Beam 6B			
0	0	0	0	0	0	0
4	1,000	0.016	0.019	0.019	0.016	0.010
5	2,000	0.027	0.034	0.034	0.030	0.019
8	2,000	0.029	0.037	0.037	0.033	0.021
10	3,000	0.038	0.050	0.050	0.045	0.028
12	4,000	0.050	0.068	0.070	0.062	0.038
15	4,000	0.054	0.072	0.074	0.065	0.041
17	5,000	0.062	0.089	0.092	0.081	0.050
19	5,000	0.074	0.108	0.112	0.098	0.060
21	6,000	0.077	0.113	0.117	0.103	0.063

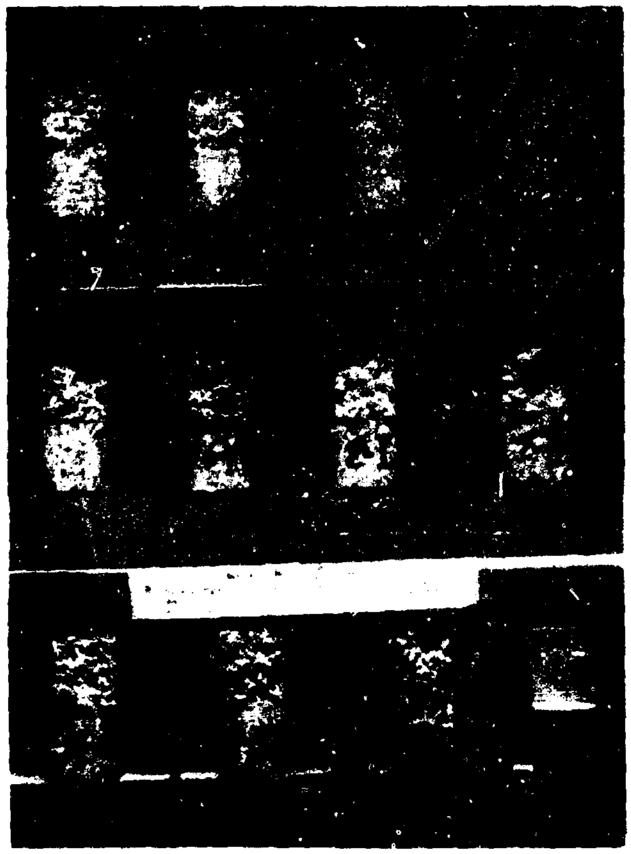
Time min	Load lb	Deflections (Midspan), in. 1	2	3	4	5
			Beam 6B (Continued)			
28	0	0.018	0.023	0.023	0.021	0.013
33	0	0.017	0.022	0.022	0.020	0.013
35	7,000	0.089	0.136	0.143	0.125	0.076
36	8,000	0.099	0.155	0.165	0.143	0.086
40	8,000	0.104	0.162	0.175	0.151	0.091
41	9,000	0.111	0.176	0.191	0.165	0.099
47	10,000	0.125	0.202	0.224	0.192	0.114
50	10,000	0.130	0.210	0.233	0.199	0.119
56	11,000	0.143	0.233	0.260	0.223	0.133
61	12,000	0.158	0.260	0.291	0.251	0.150
65	12,000	0.240	0.435	0.484	0.396	0.220
70	12,000			Failure		
			Beam 7B			
0	0	0	0	0	0	0
7	1,000	0.012	0.016	0.016	0.016	0.010
11	2,000	0.022	0.030	0.030	0.029	0.019
13	2,000	0.023	0.032	0.033	0.034	0.021
16	3,000	0.031	0.044	0.046	0.045	0.029
18	4,000	0.042	0.061	0.065	0.062	0.040
22	4,000	0.046	0.067	0.071	0.067	0.043
23	5,000	0.055	0.084	0.091	0.085	0.053
24	6,000	0.079	0.130	0.154	0.132	0.079
32	6,000	0.088	0.144	0.168	0.146	0.086
35	0	0.021	0.036	0.040	0.036	0.043
39	0	0.021	0.034	0.038	0.035	0.022
49	6,840			Failure		
54	0	0.072	0.128	0.190	0.134	0.073
			Beam 8B			
0	0	0	0	0	0	0
7	1,000	0.019	0.022	0.022	0.019	0.012
9	2,000	0.031	0.038	0.043	0.036	0.023
12	2,300	0.033	0.040	0.045	0.038	0.024
13	3,000	0.043	0.059	0.070	0.059	0.037
15	3,580			Failure		

Table 15

Moment-Curvature Relations, Series B

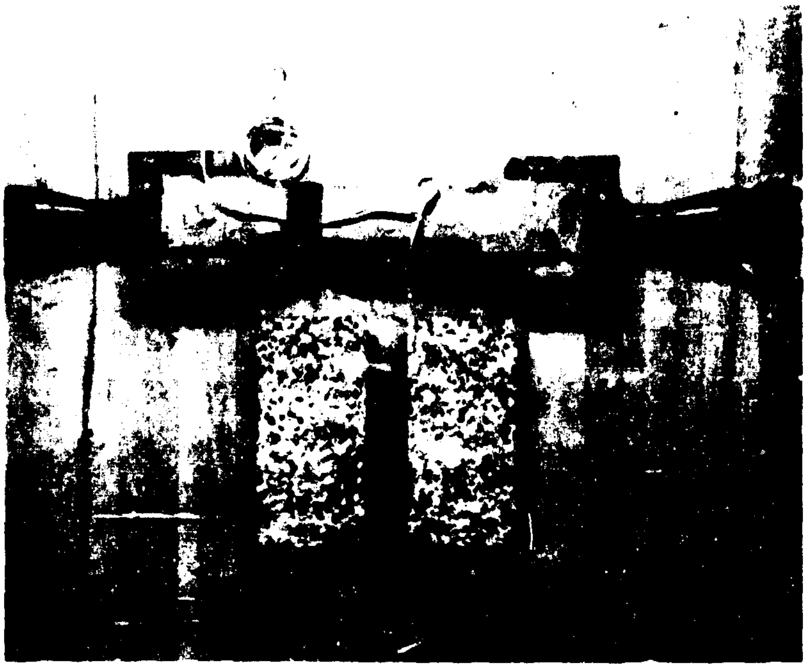
Moment ft-lb	$\frac{\epsilon_r - \epsilon_c}{d}$ 10^{-6} Radians	$\frac{\epsilon_{8,9,12,13} - \epsilon_c}{a_1}$ 10^{-6} Radians	$\frac{\epsilon_{10,11,14,15} - \epsilon_c}{a_2}$ 10^{-6} Radians	Average ϕ 10^{-6} Radians
Beam 1B (d = 8.00 in.; a_1 = 8.10 in.; a_2 = 8.95 in.)				
0	0	0	0	
2,000	62.0	78.1	119.2	
4,000	142.6	207.5	223.8	
6,000	202.8	301.0	310.3	
0	25.3	211.2	198.1	NA
8,000	273.3	398.0	403.8	
10,000	331.6	470.6	488.3	
12,000	481.1	1419.0	1504.6	
0	--	--	--	
Beam 6B (d = 8.00 in.; a_1 = 8.00 in.; a_2 = 8.85 in.)				
0	0	0	0	0
2,000	42.0	41.1	45.5	42.9
4,000	96.3	90.6	100.8	95.9
6,000	159.5	147.3	166.1	157.6
0	21.1	17.9	20.3	19.8
8,000	248.7	228.6	264.1	247.1
10,000	324.9	300.0	346.0	323.6
12,000	409.5	366.5	314.5	363.5
Beam 7B (d = 7.90 in.; a_1 = 7.90 in.; a_2 = 8.75 in.)				
0	0	0	0	0
2,000	42.4	37.6	41.7	40.6
4,000	91.8	86.3	92.9	90.3
6,000	329.7	600.3	724.8	551.6
0	84.3	320.1	277.5	227.3

Note: NA = not applicable.



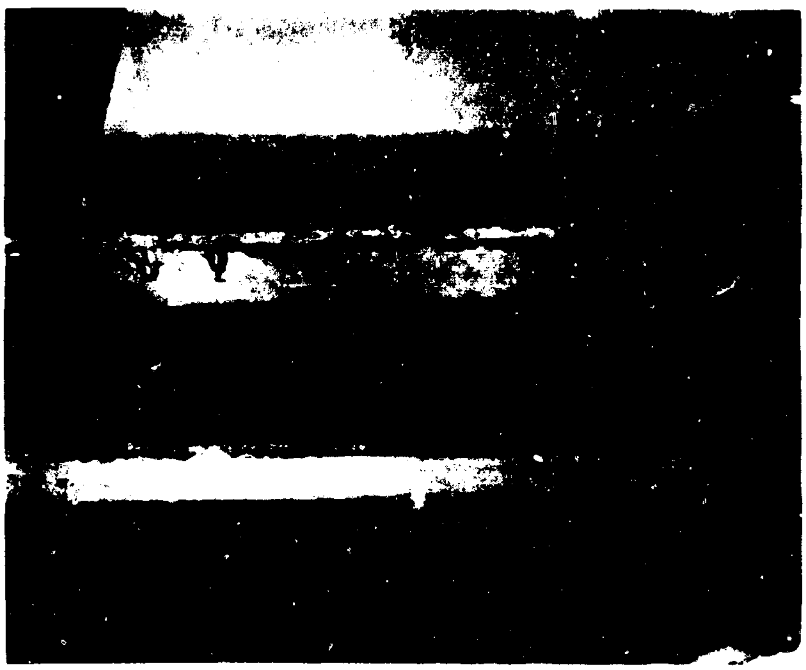
Photograph 1. Typical epoxy resin concrete specimens after testing

Photograph 2. Flexural test on 2- by 2- by 11-1/4-in. prisms

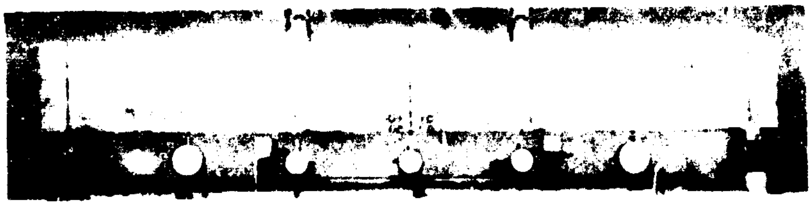


Photograph 3. Typical specimen used for direct tension and tensile splitting tests

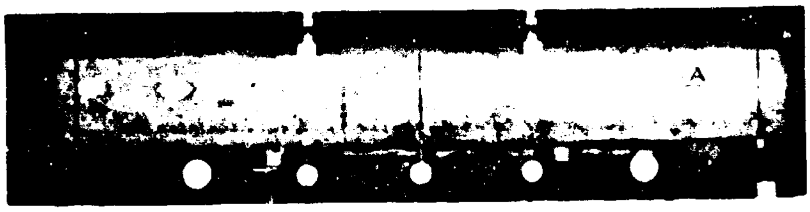
Photograph 4. Setup for beam tests



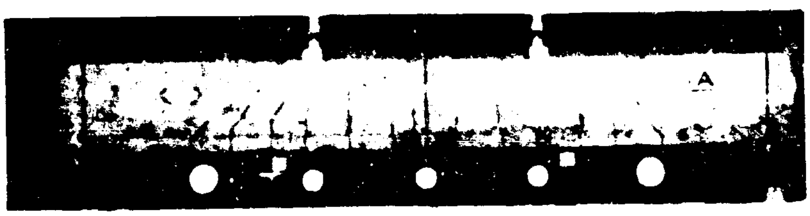
Photograph 5. Shrinkage cracks in polyester resin-limestone aggregate concrete



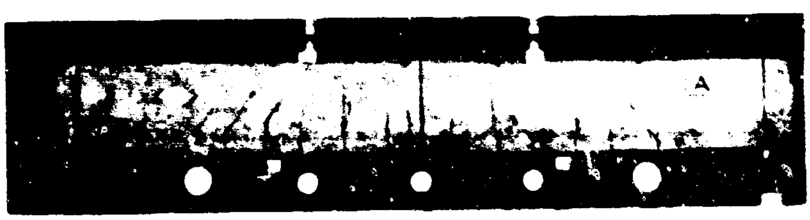
Total load, 4000 lb



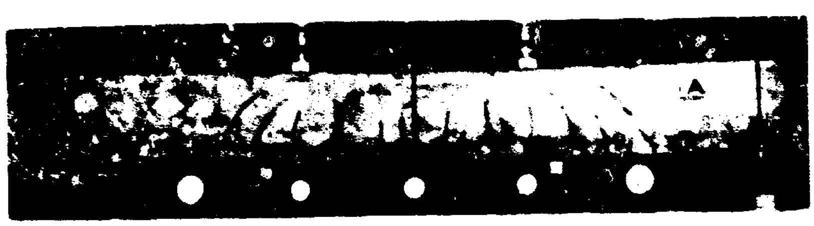
b. Total load, 6000 lb



c. Total load, 9000 lb



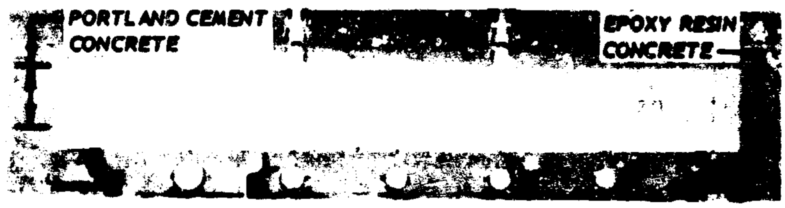
d. Total load, 11,000 lb



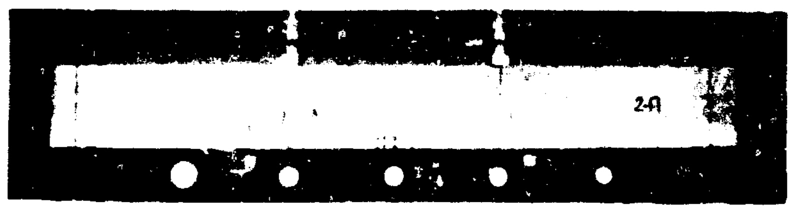
e. Total load, 13,000 lb

f. [illegible] failure

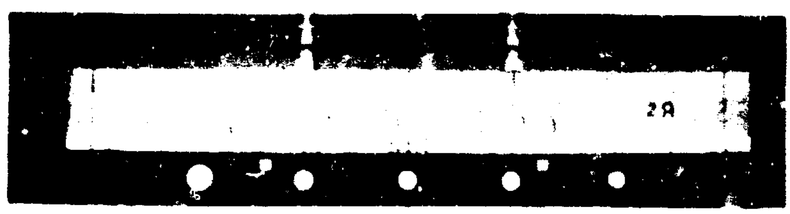
Photograph 6. Crack [illegible], beam 1A (reference beam)



a. Total load, 6500 lb

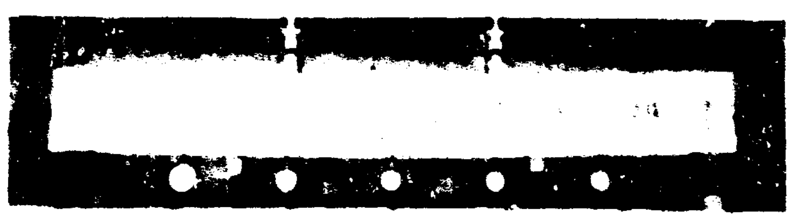


b. Total load, 8000 lb



c. Total load, 10,000 lb

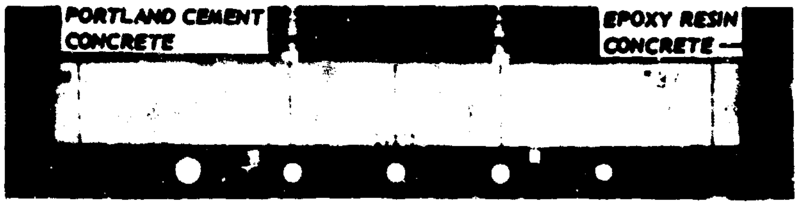
d. Total load, 14,000 lb



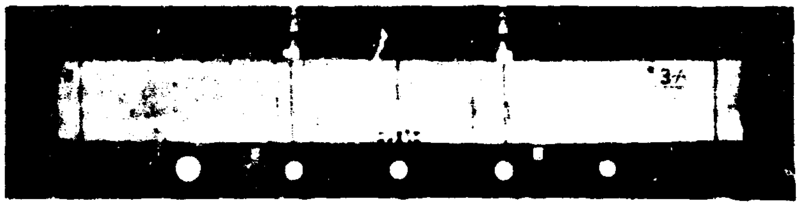
e. Total load, 15,000 lb

f. Total load, 15,300 lb (ultimate)

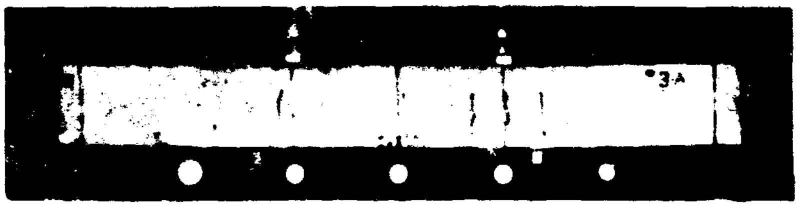
Photograph 7. Crack pattern, beam 2A



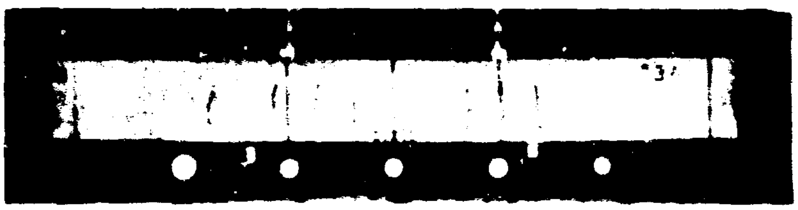
a. Total load, 3000 lb



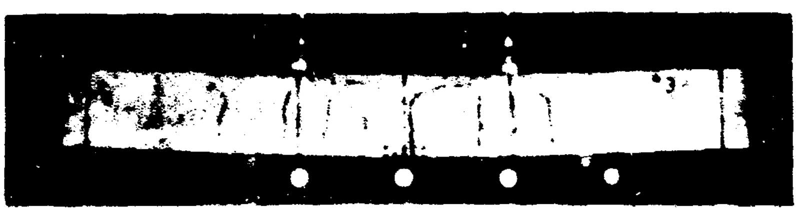
b. Total load, 4000 lb



c. Total load, 6000 lb



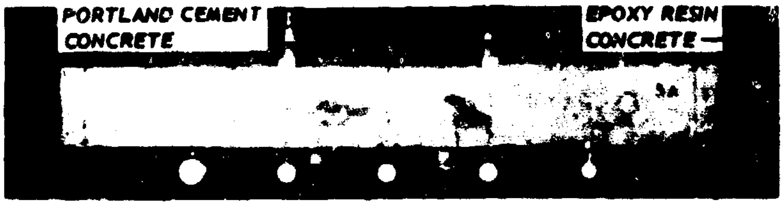
d. Total load, 7000 lb



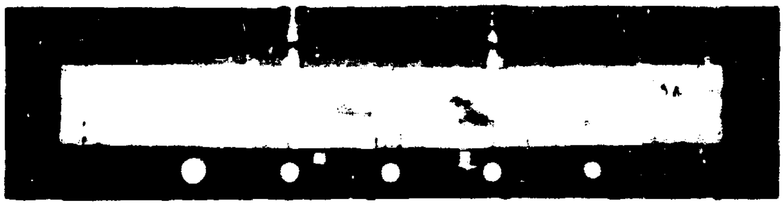
e. Total load, 8100 lb (ultimate)

Photograph 8. Crack pattern, beam 3A

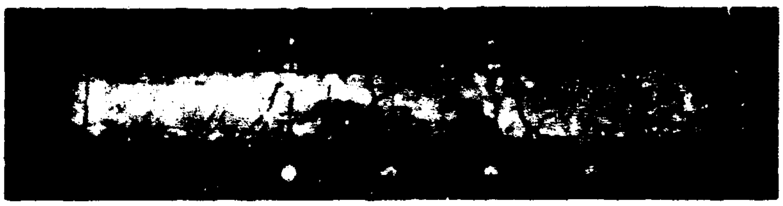
Photograph 9. Crack pattern, beam 4A, after failure at 6140 lb



a. Total load, 6000 lb



b. Total load, 8000 lb



c. Total load, 10,000 lb

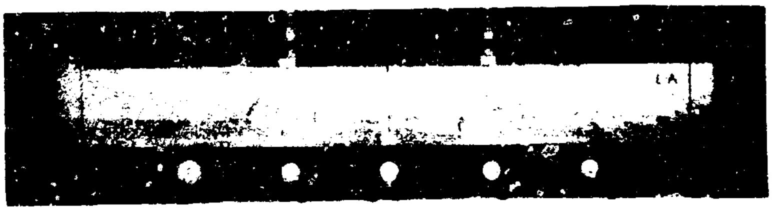
d. Total load, 14,000 lb

e. Total load, 16,000 lb

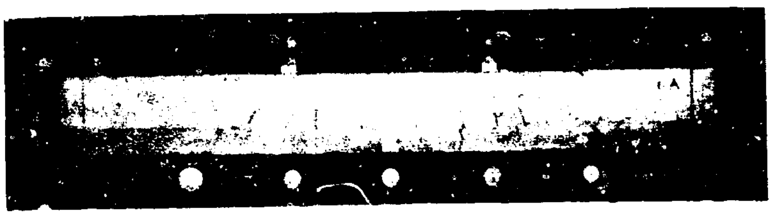
f. After failure

Photograph 10. Crack pattern, beam 5A

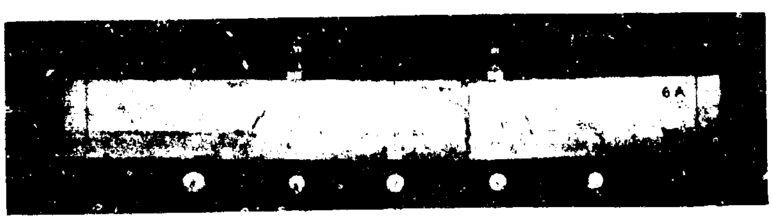
a. Total load, 9000 lb



b. Total load, 12,000 lb



c. Total load, 13,000 lb



d. Total load, 14,500 lb (ultimate)

Photograph 11. Crack pattern, beam 6A

a. Total load, 5500 lb

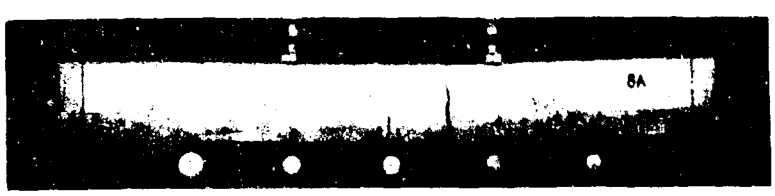
b. Total load, 8000 lb

c. Total load, 9000 lb (ultimate)

Photograph 12. Crack pattern, beam 7A

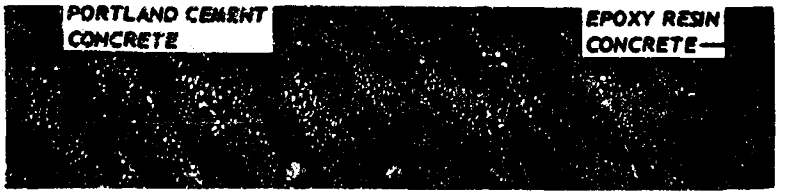
a. Total load, 4000 lb

b. Total load, 5000 lb

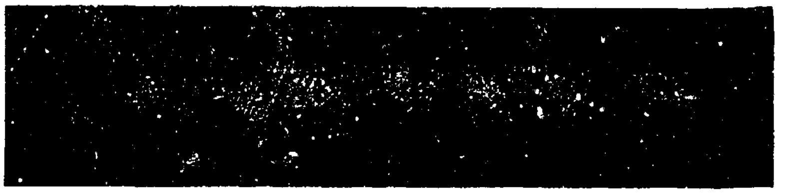


c. Total load, 5500 lb (ultimate)

Photograph 13. Crack pattern, beam 8A

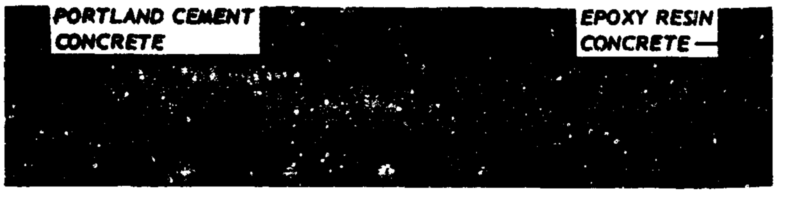


a. Total load, 3000 lb

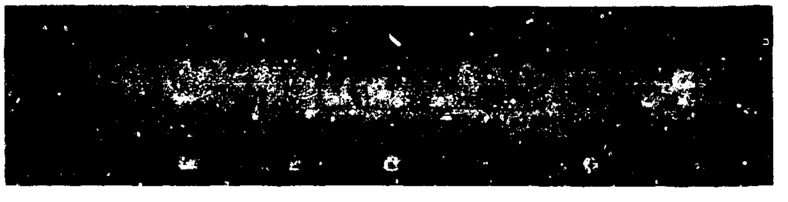


b. After failure

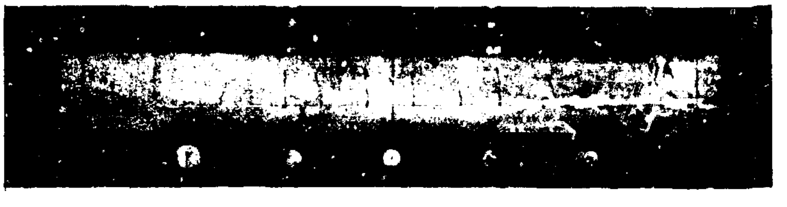
Photograph 14. Crack pattern, beam 9A



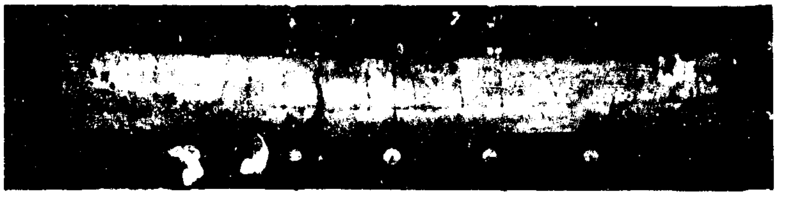
a. Total load, 3000 lb



b. Total load, 3500 lb

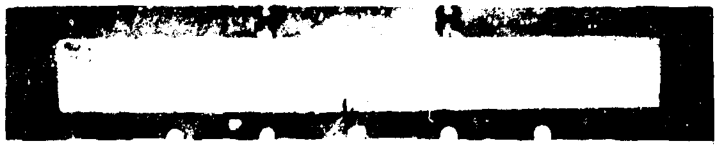


c. Total load, 4000 lb

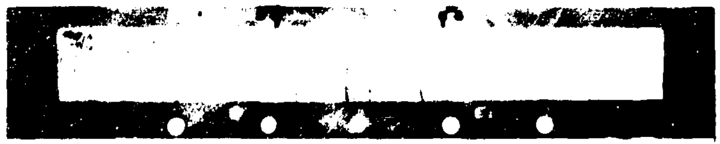


d. After failure

Photograph 15. Crack pattern, beam 11A

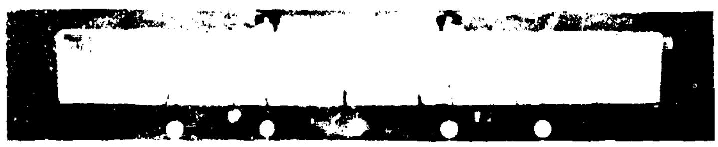


a. Total load, 3000 lb



b. Total load, 4000 lb

c. Total load, 6000 lb



d. Total load, 7000 lb

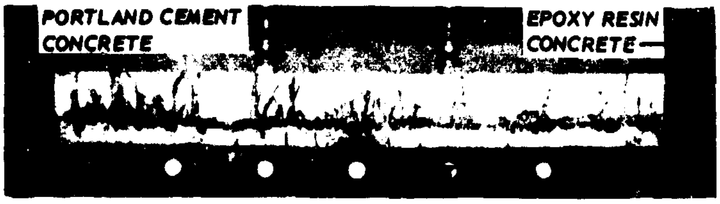
e. Total load, 10,000 lb

f. Total load, 11,000 lb

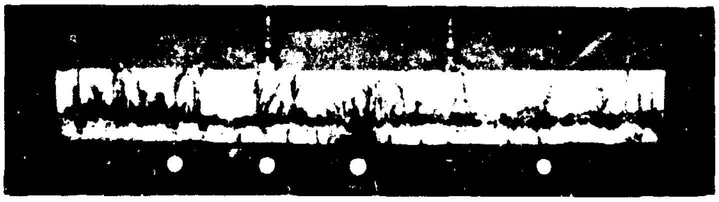
g. Total load, 12,000 lb

h. Total load, 12,280 lb (ultimate)

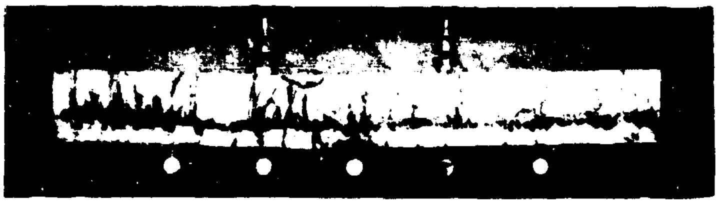
Photograph 16. Crack pattern, beam 1B



a. Total load, 9000 lb

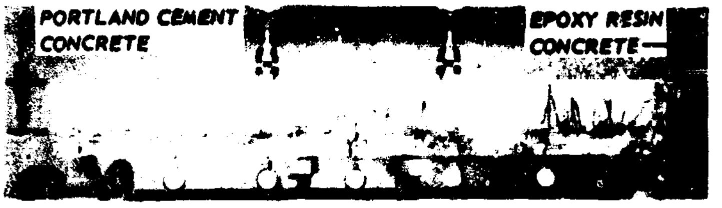


b. Total load, 10,000 lb

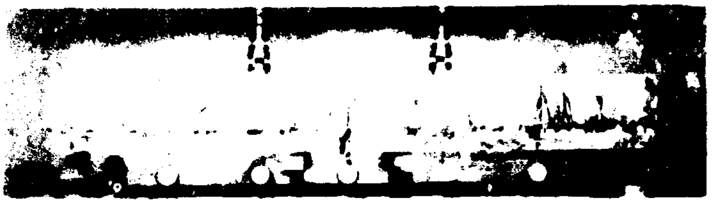


c. Total load, 12,000 lb (ultimate)

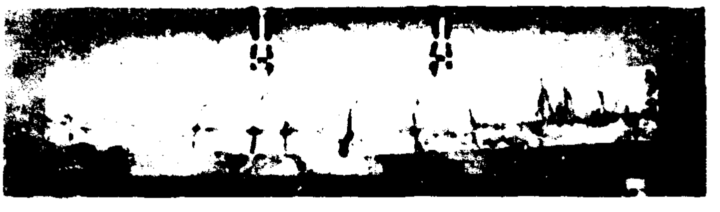
Photograph 17. Crack pattern, beam 6B



a. Total load, 6000 lb

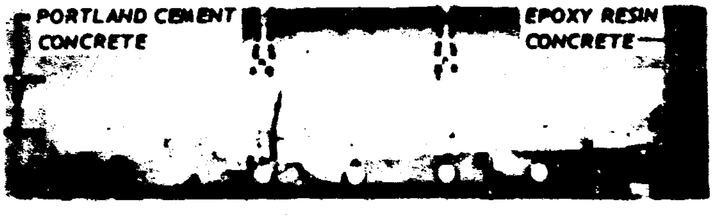


b. Total load, 6800 lb

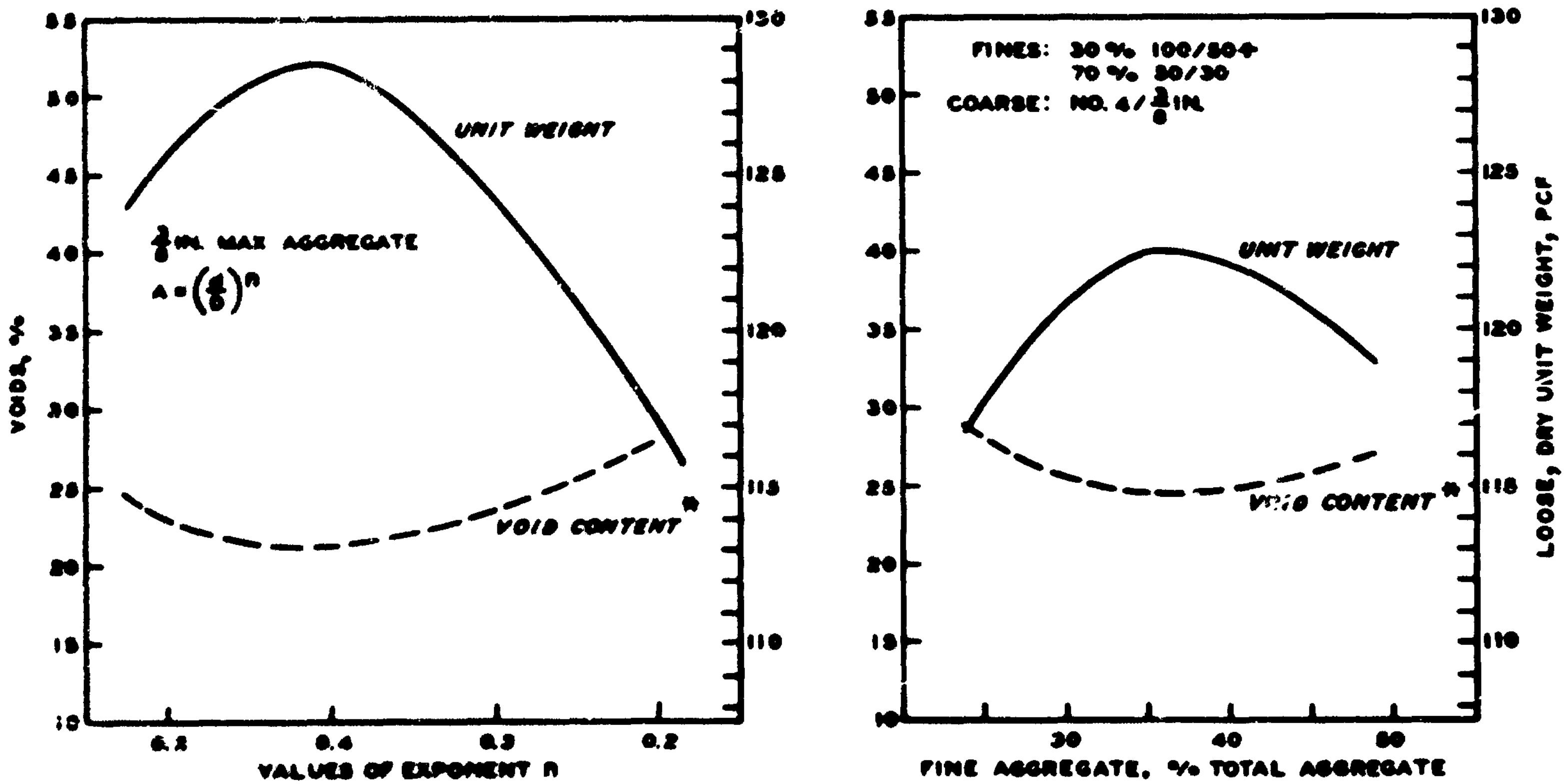


c. After failure at 6840 lb

Photograph 18. Crack pattern, beam 7B



Photograph 19. Crack pattern, beam 8B, after failure



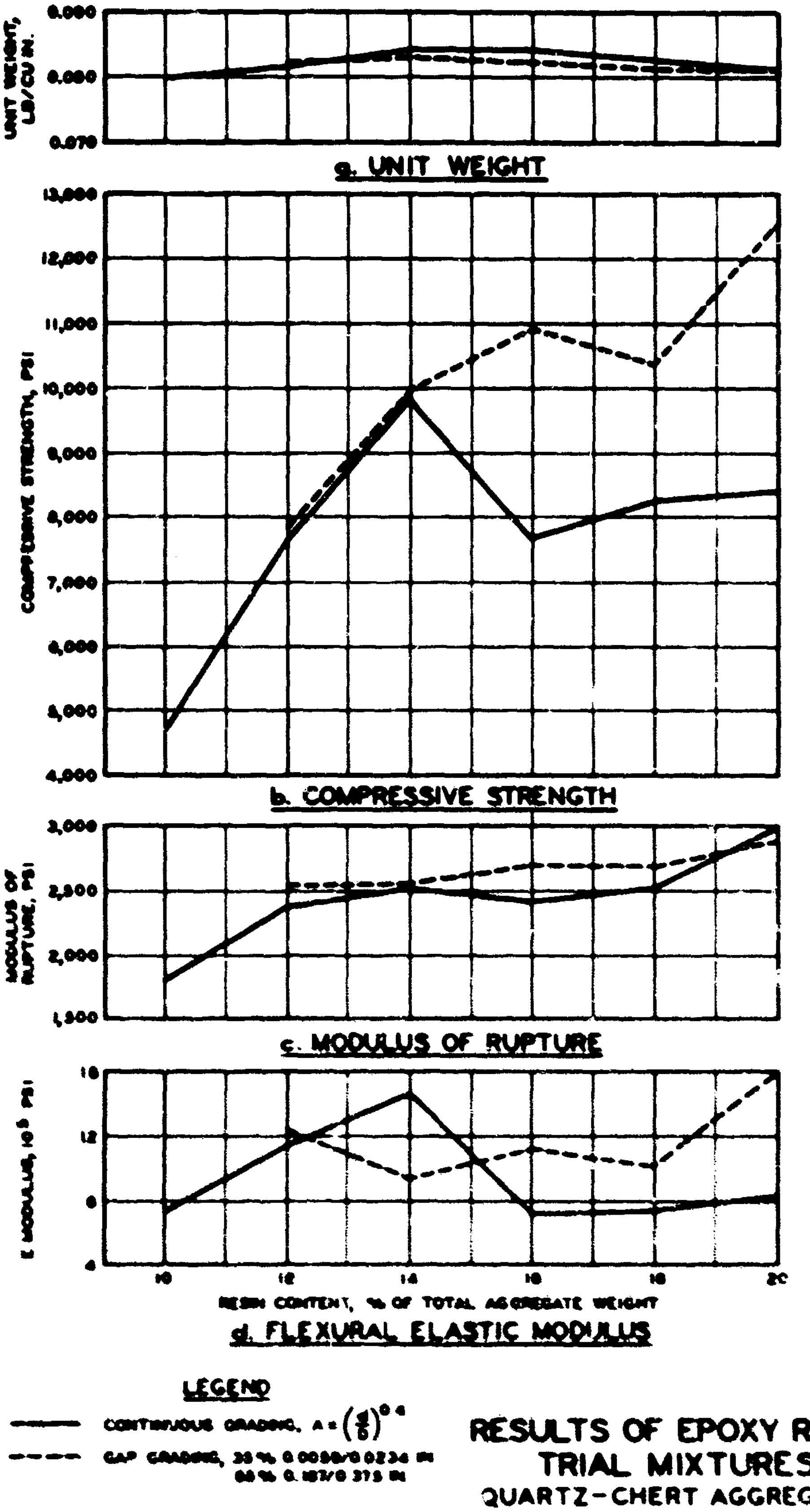
a. CONTINUOUS GRADING

b. GAP GRADING

* FOR A DENSITY OF 163 PCF.

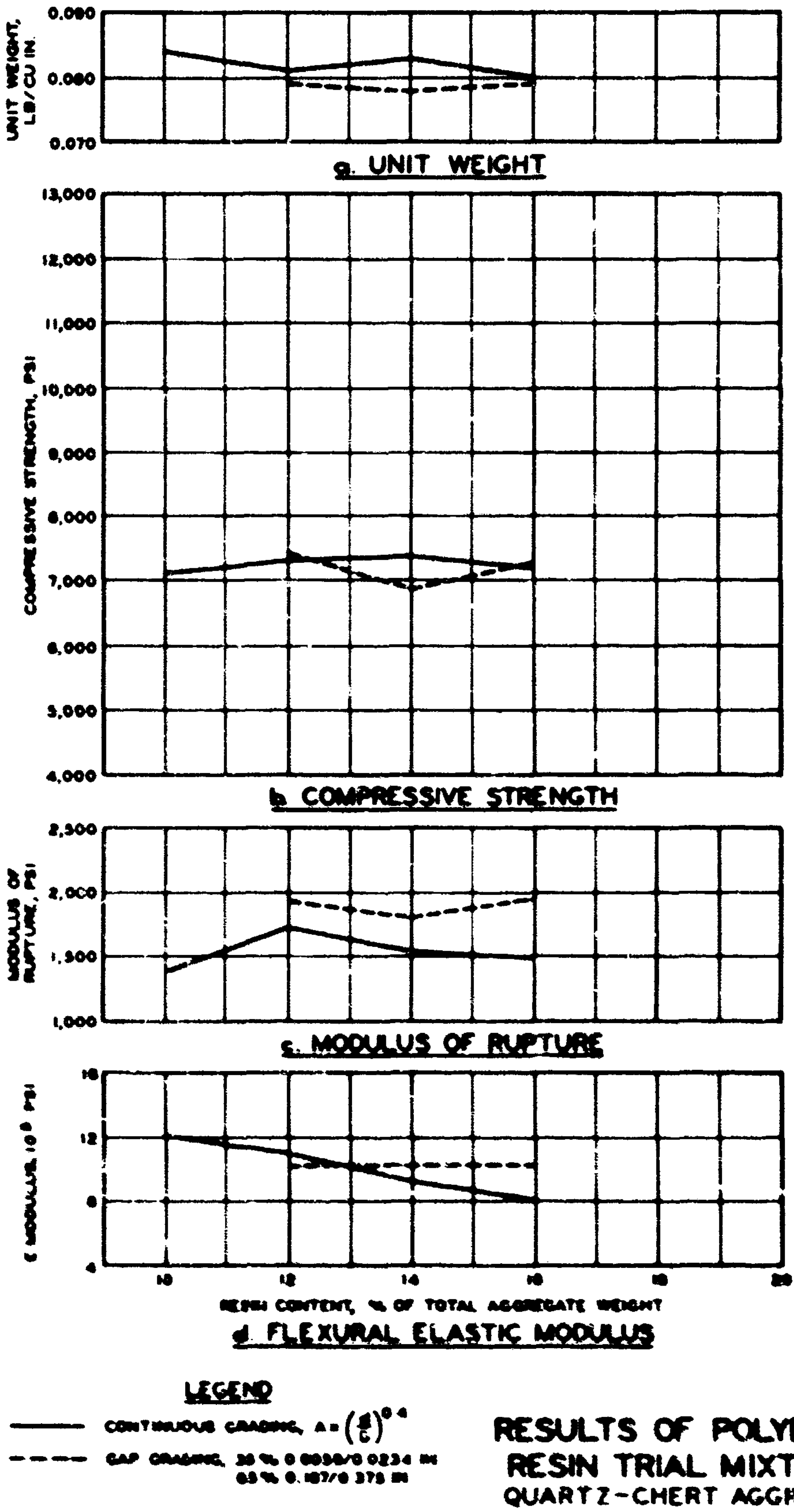
GRADING - BULK DENSITY RELATION

QUARTZ-CHERT AGGREGATE

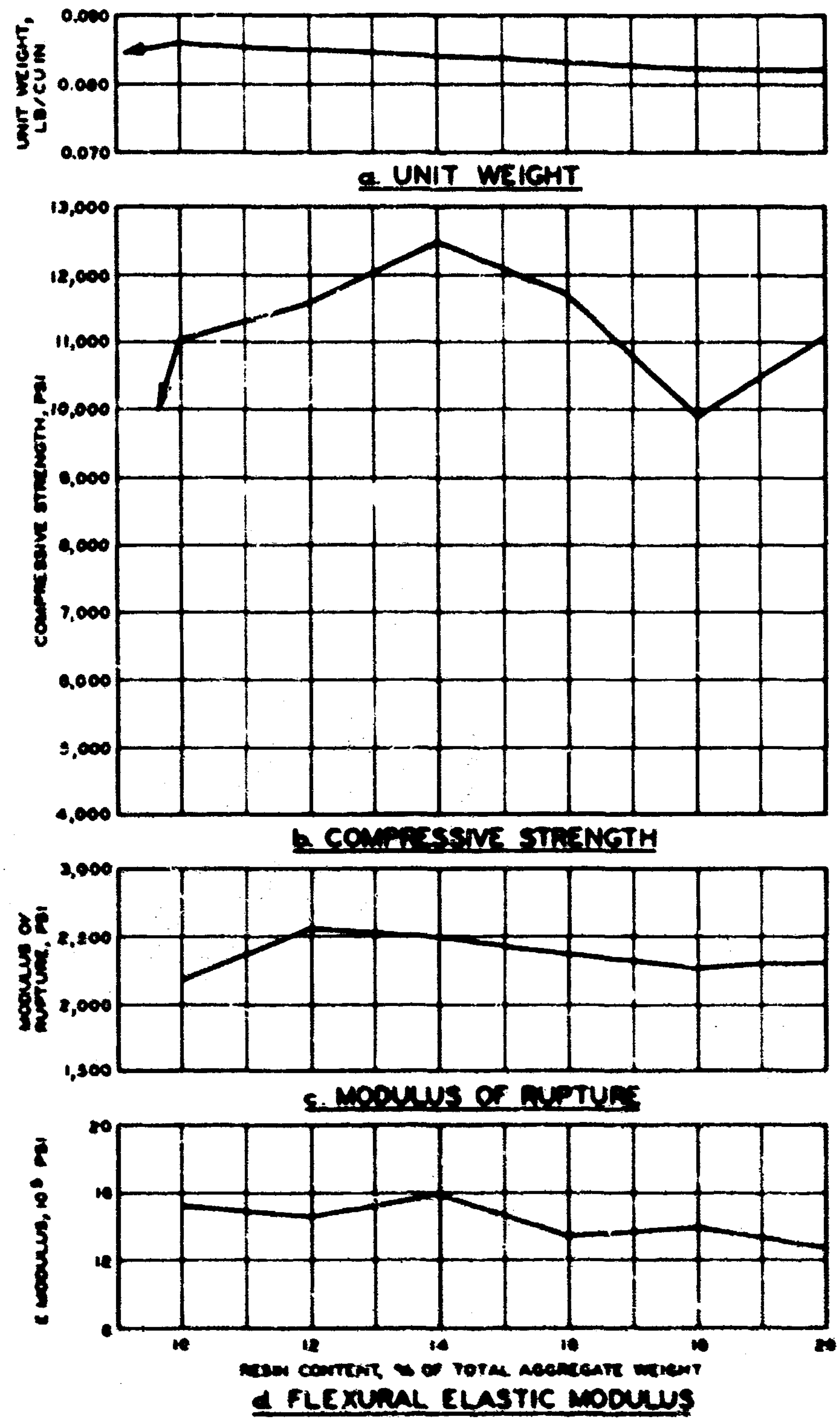


RESULTS OF EPOXY RESIN TRIAL MIXTURES
QUARTZ-CHERT AGGREGATE

PLATE 2



RESULTS OF POLYESTER RESIN TRIAL MIXTURES
QUARTZ-CHERT AGGREGATE



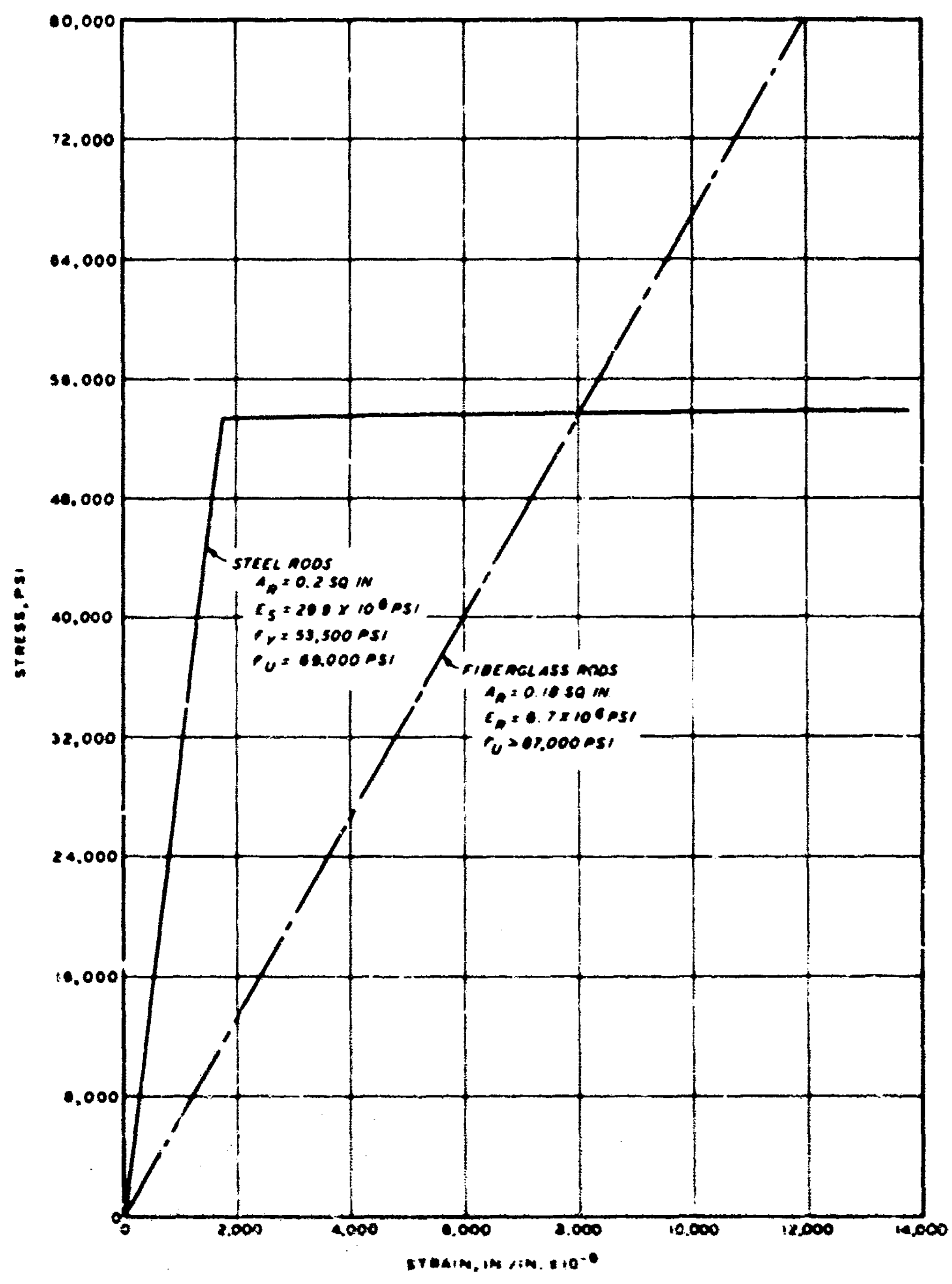
LEGEND

CONTINUOUS GRADING

RESULTS OF POLYESTER RESIN TRIAL MIXTURES

LIMESTONE AGGREGATE

PLATE 4



STRESS-STRAIN CURVES
OF REINFORCING BARS

PLATE 5

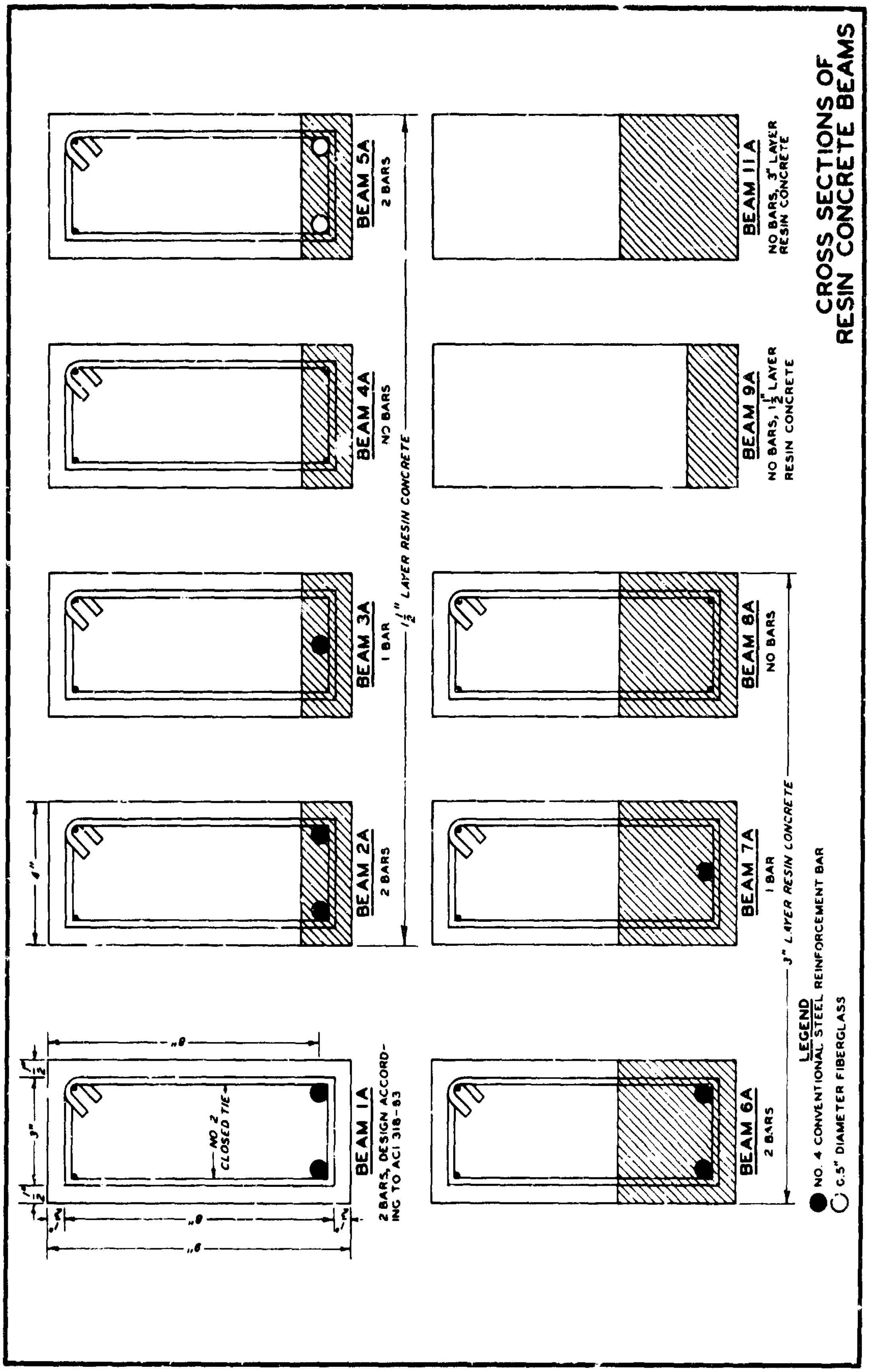
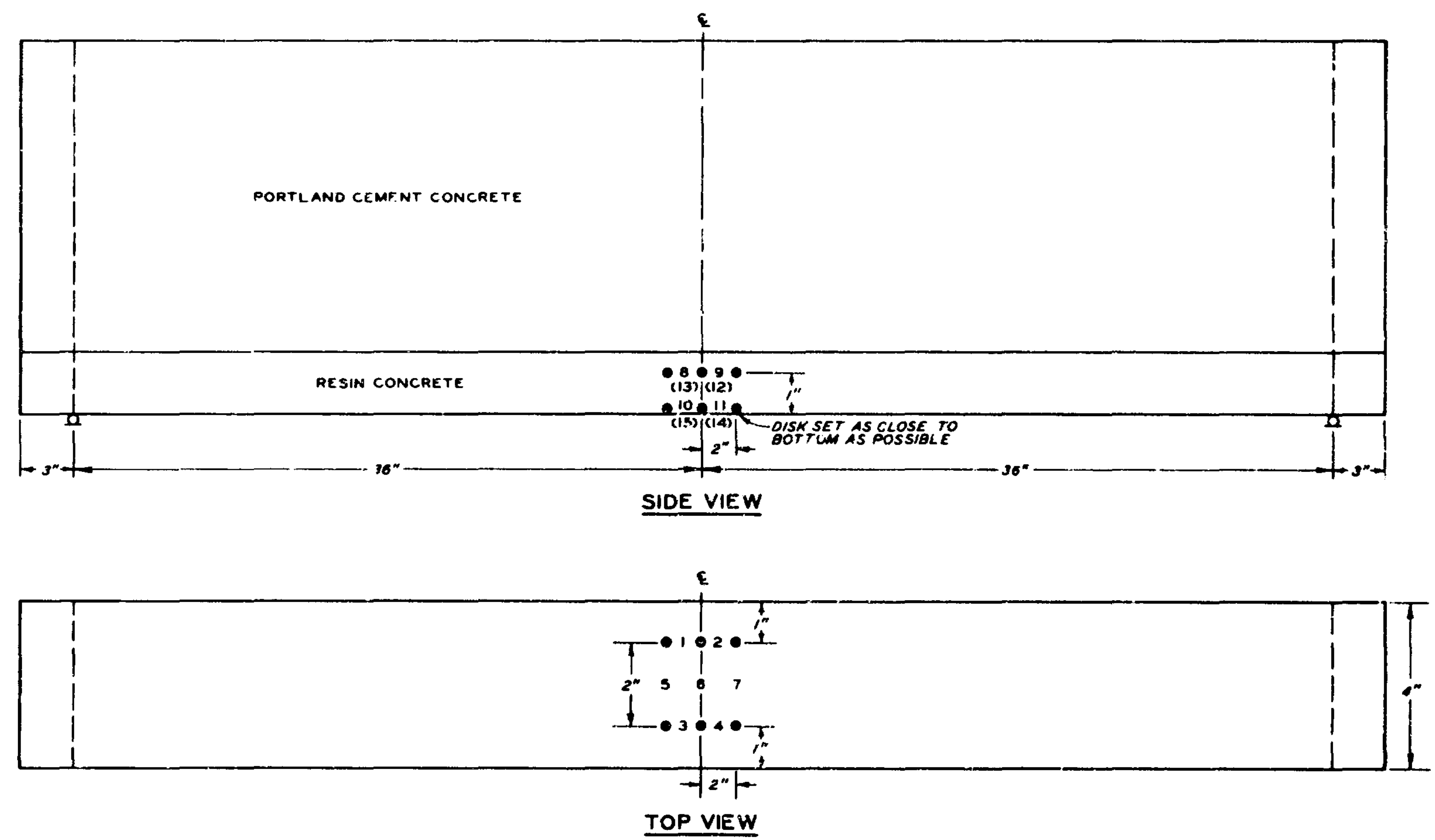
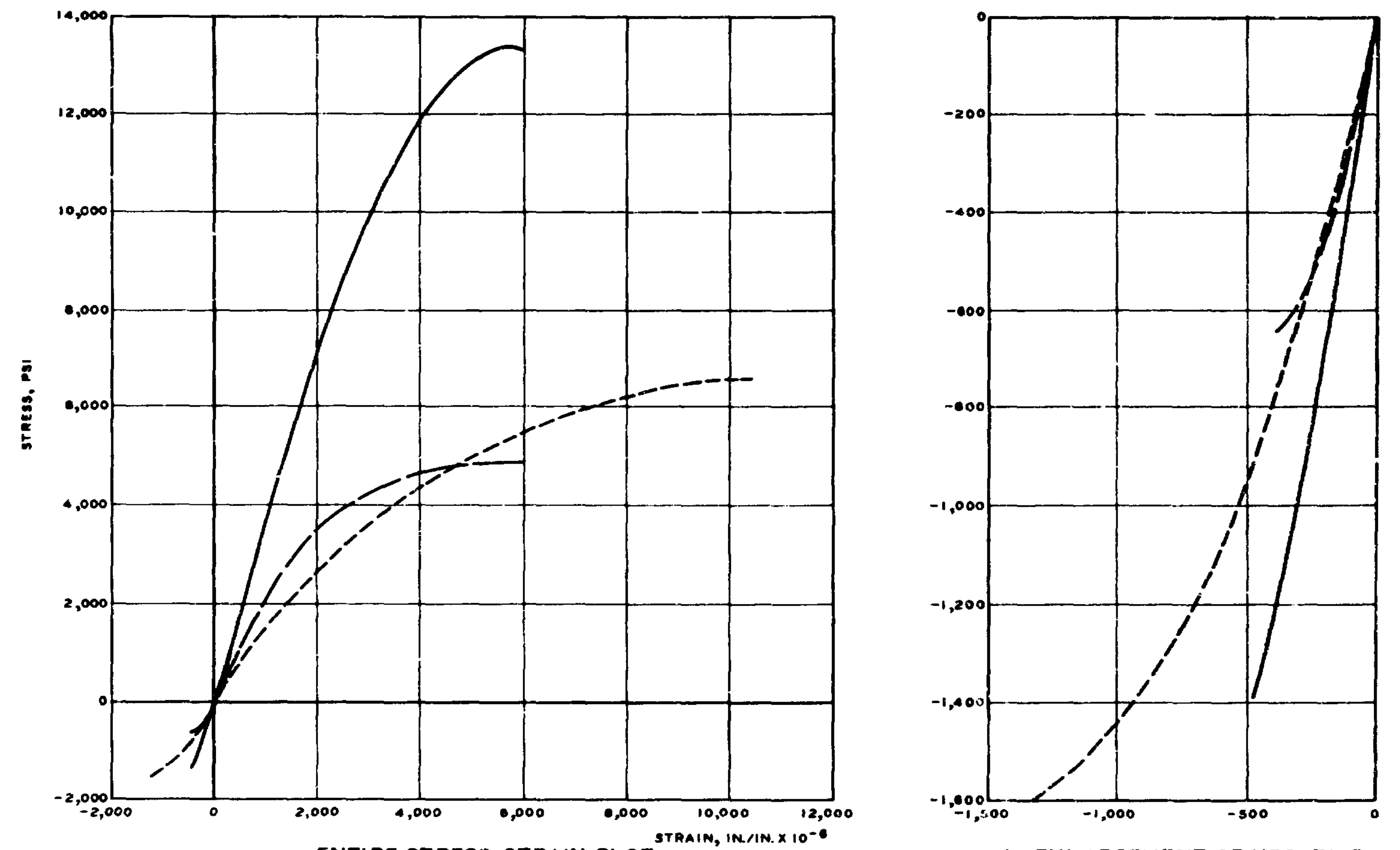

CROSS SECTIONS OF RESIN CONCRETE BEAMS
BEAM 1A
2 BARS, DESIGN ACCORDING TO ACI 318-83
NO 2 CLOSED TIE
BEAM 2A
2 BARS
BEAM 3A
1 BAR
BEAM 4A
NO BARS
BEAM 5A
2 BARS
1 1/2" LAYER RESIN CONCRETE
BEAM 6A
2 BARS
BEAM 7A
1 BAR
BEAM 8A
NO BARS
BEAM 9A
NO BARS, 1 1/2" LAYER RESIN CONCRETE
BEAM 11A
NO BARS, 3" LAYER RESIN CONCRETE
3" LAYER RESIN CONCRETE
LEGEND
NO. 4 CONVENTIONAL STEEL REINFORCEMENT BAR
0.5" DIAMETER FIBERGLASS

PLATE 6



LOCATION OF STRAIN MEASURING DISKS



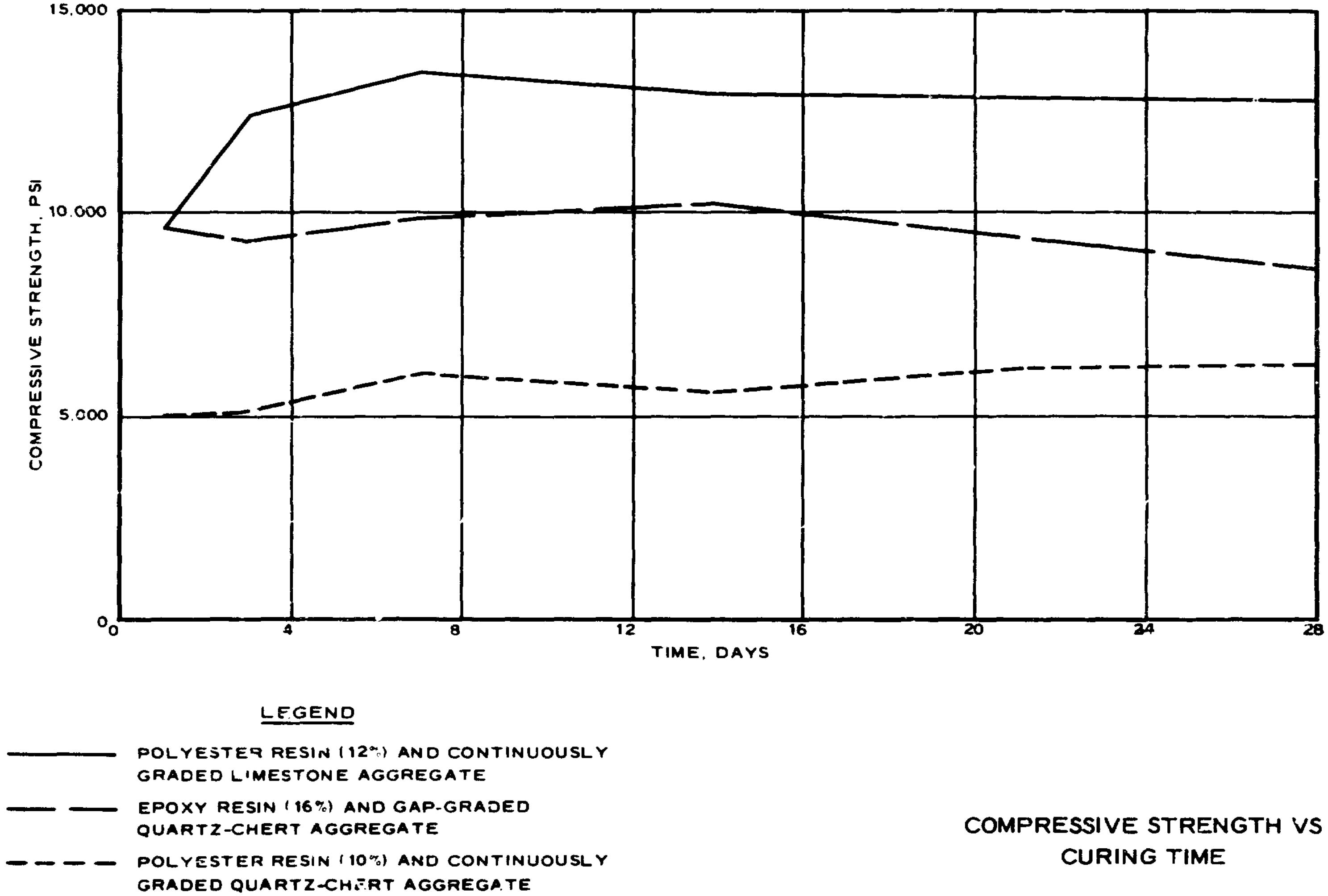
LEGEND

- ——— POLYESTER RESIN AND CONTINUOUS-GRADED LIMESTONE AGGREGATE
- — — POLYESTER RESIN AND CONTINUOUS-GRADED QUARTZ-CHERT AGGREGATE
- - - - EPOXY RESIN AND GAP-GRADED QUARTZ-CHERT AGGREGATE

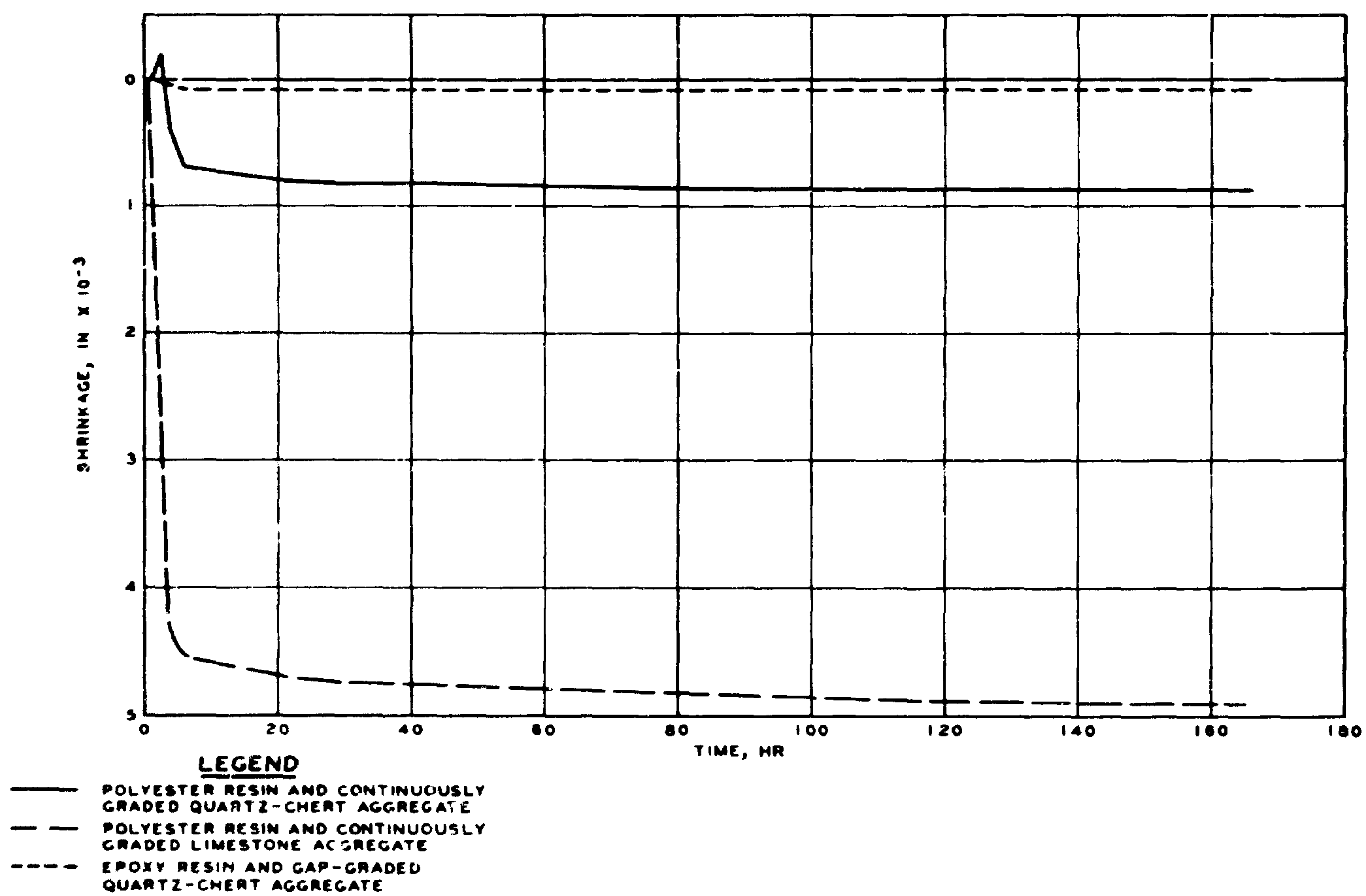
a. ENTIRE STRESS-STRAIN PLOT

b. ENLARGEMENT OF NEGATIVE PORTION OF PLOT

TENSILE STRESS-STRAIN CHARACTERISTICS OF SELECTED RESIN CONCRETES

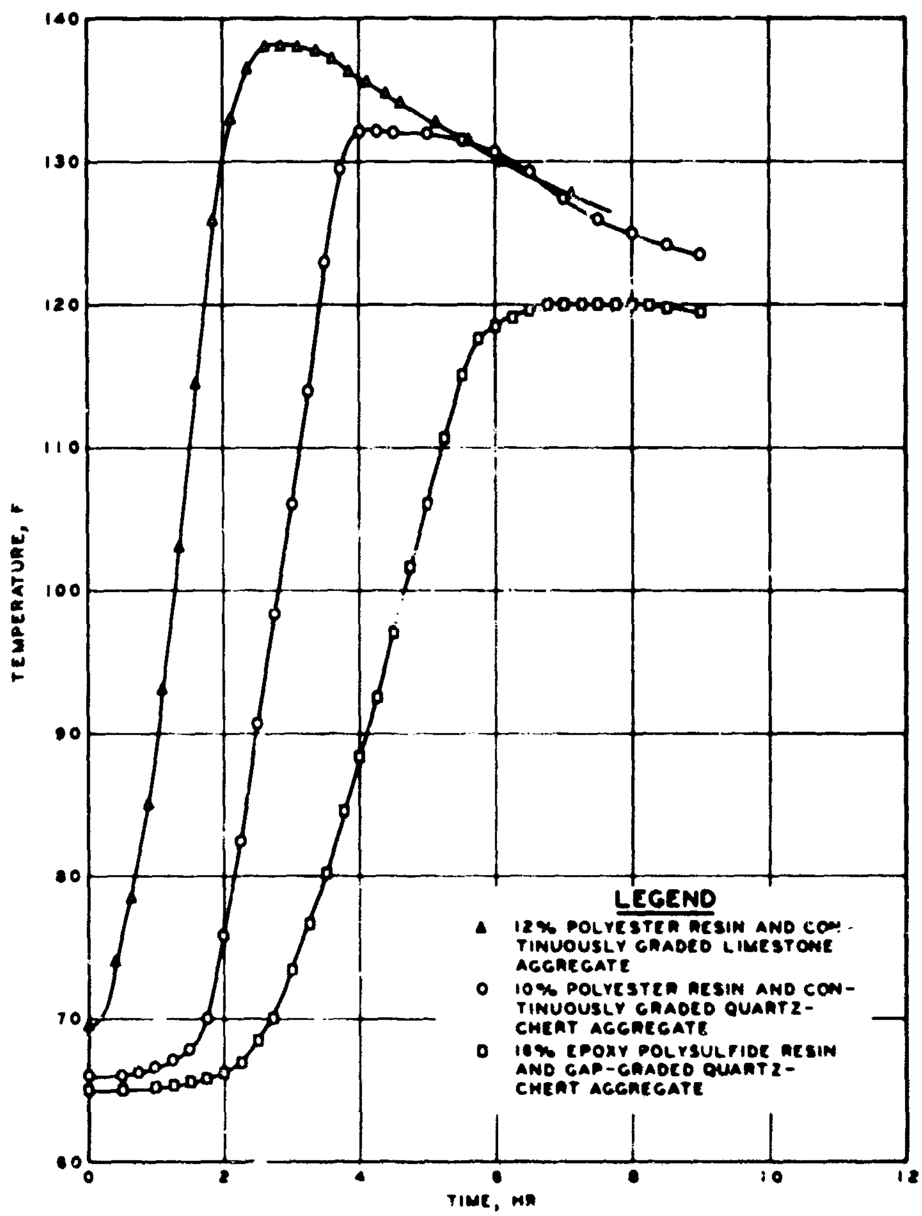


COMPRESSIVE STRENGTH VS
CURING TIME

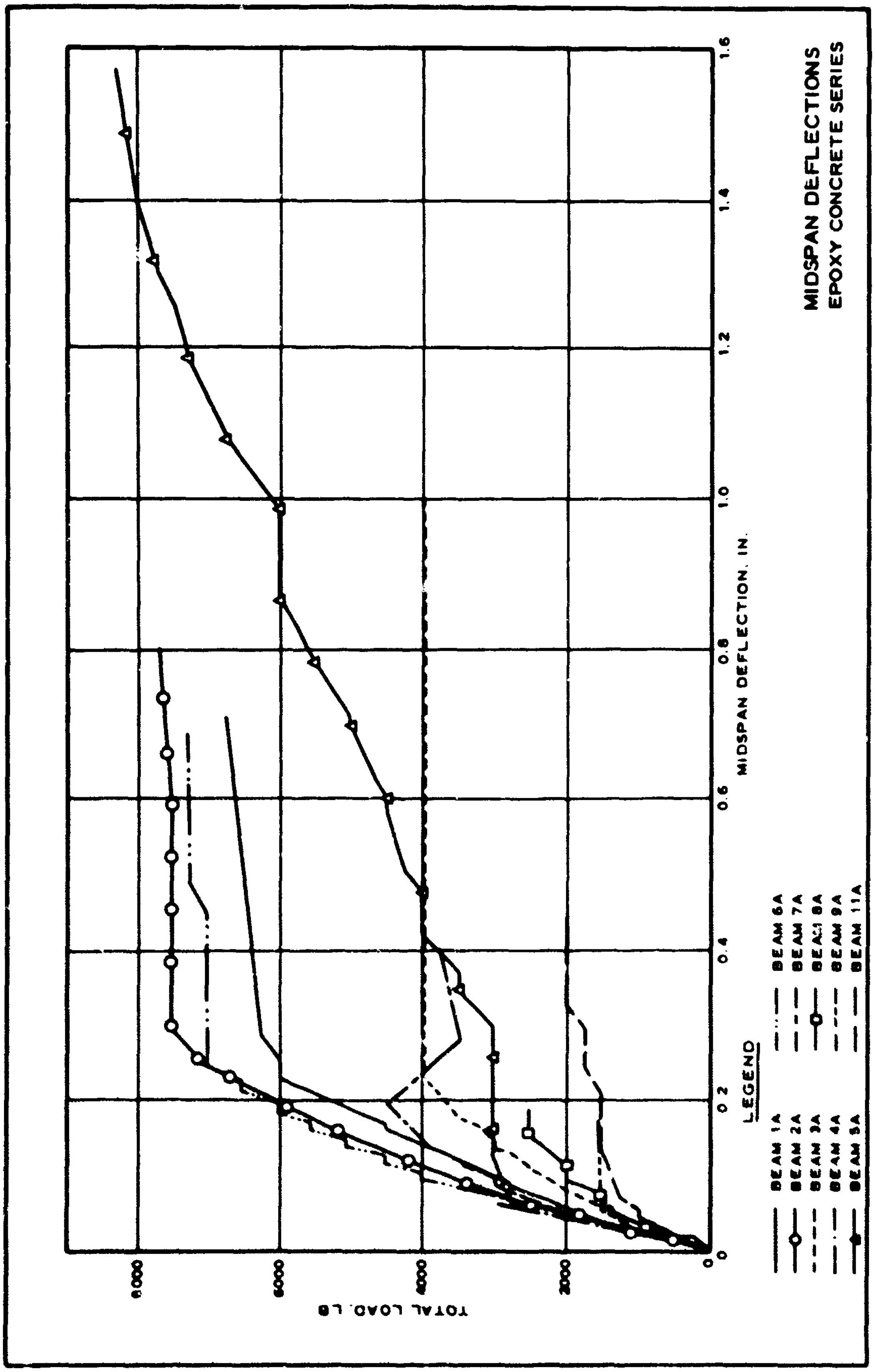


SHRINKAGE CURVES

PLATE 10

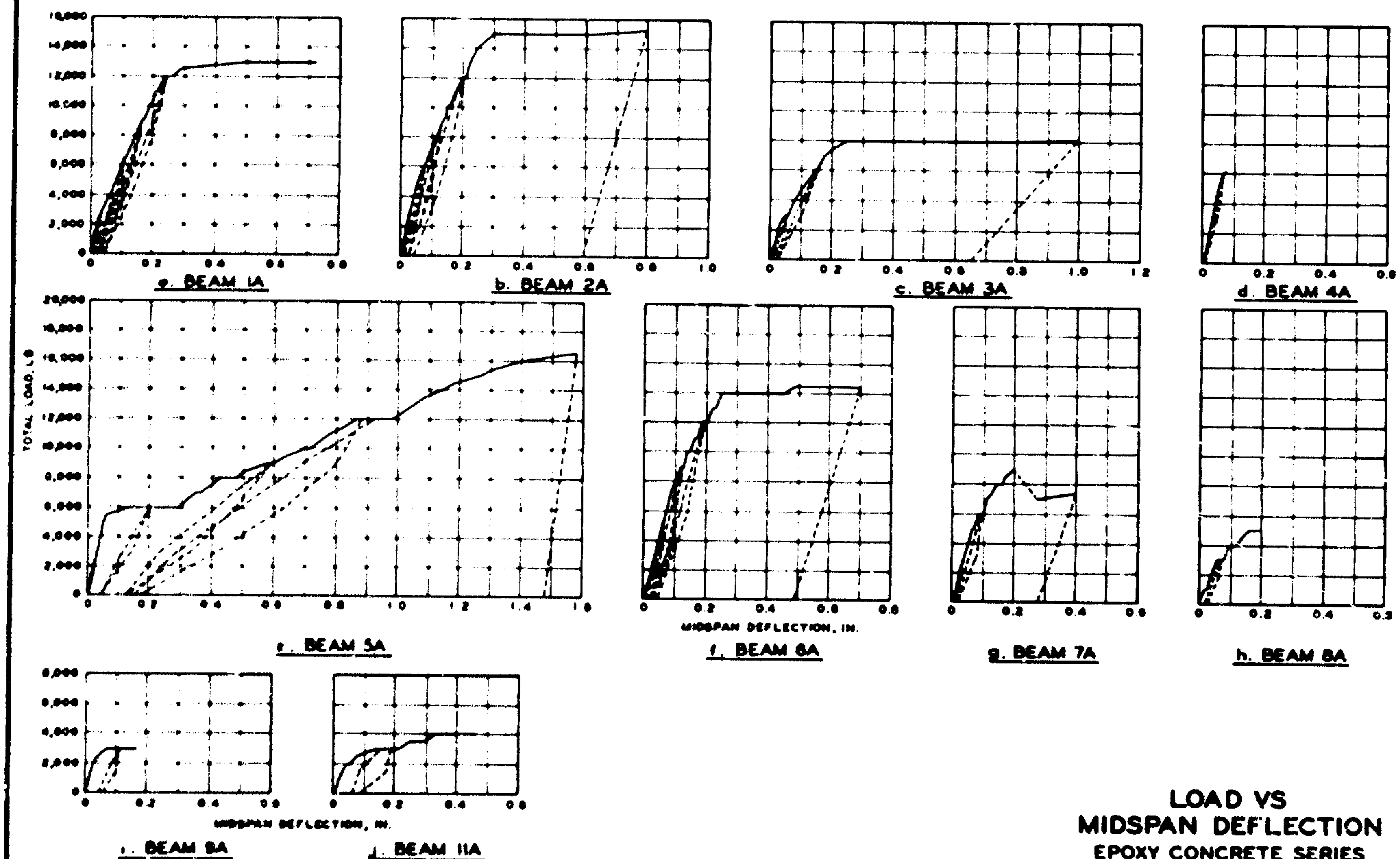


TEMPERATURE RISE IN RESIN CONCRETE IN 1-QUART VACUUM BOTTLES

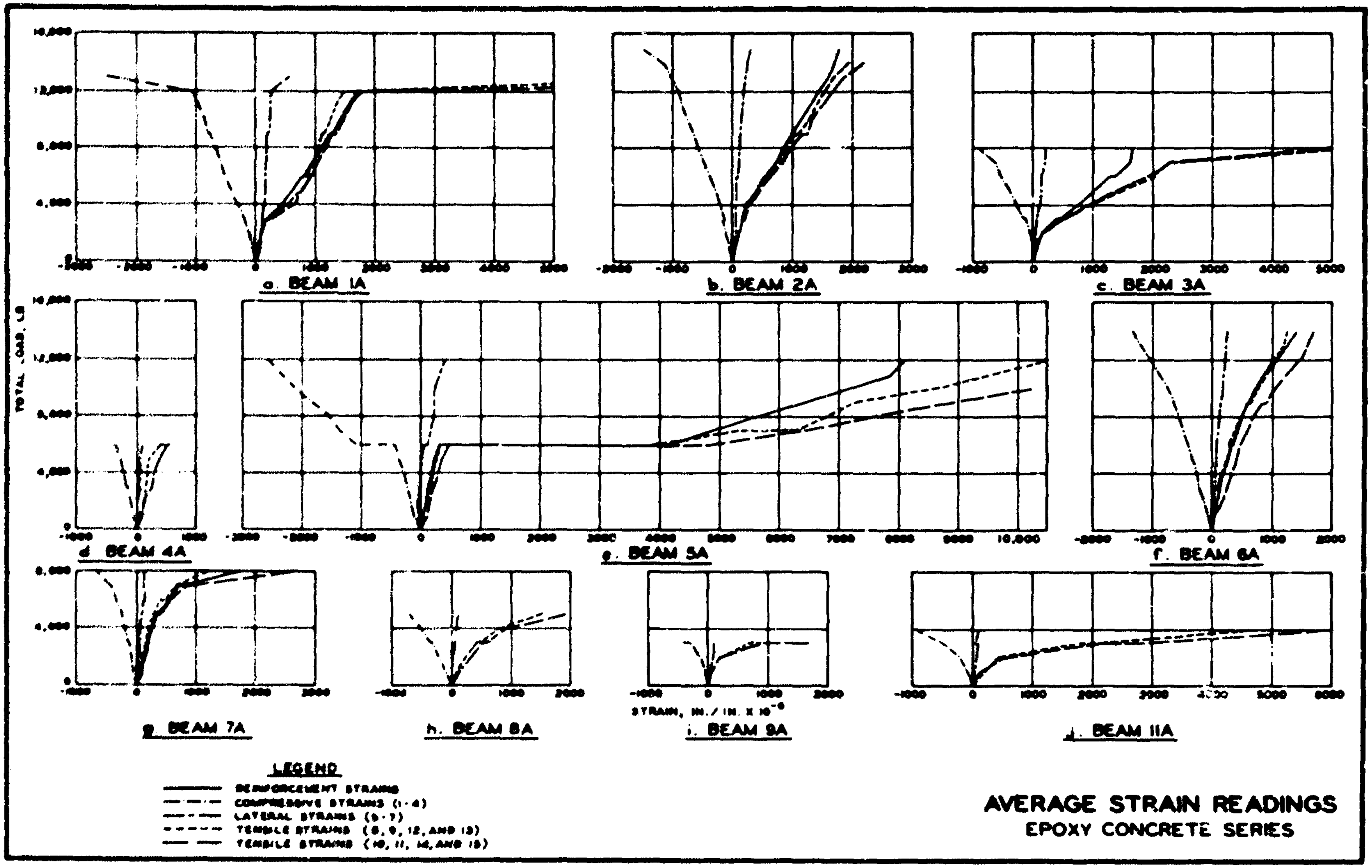


MIDSPAN DEFLECTIONS
EPOXY CONCRETE SERIES
MIDSPAN DEFLECTION, IN.
TOTAL LOAD, LB
0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
2000
4000
6000
8000
LEGEND
BEAM 1A
BEAM 2A
BEAM 3A
BEAM 4A
BEAM 5A
BEAM 6A
BEAM 7A
BEAM 8A
BEAM 9A
BEAM 11A

PLATE 12

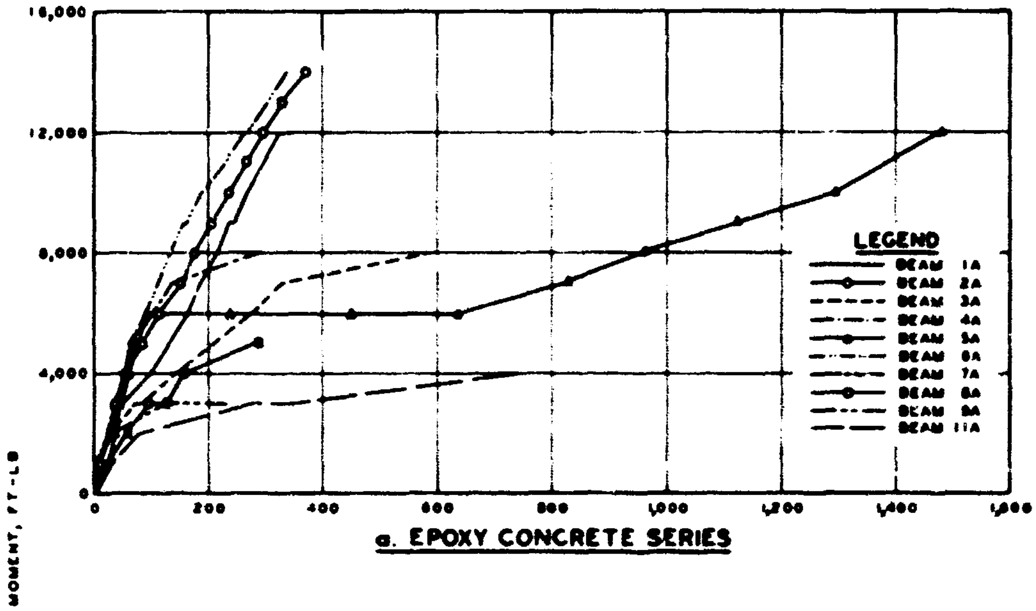


LOAD VS
MIDSPAN DEFLECTION
EPOXY CONCRETE SERIES

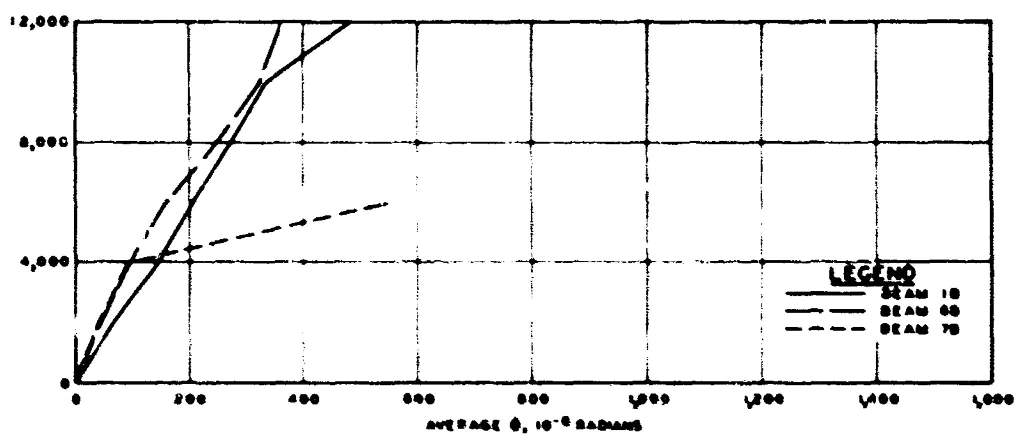


AVERAGE STRAIN READINGS
EPOXY CONCRETE SERIES

PLATE 14

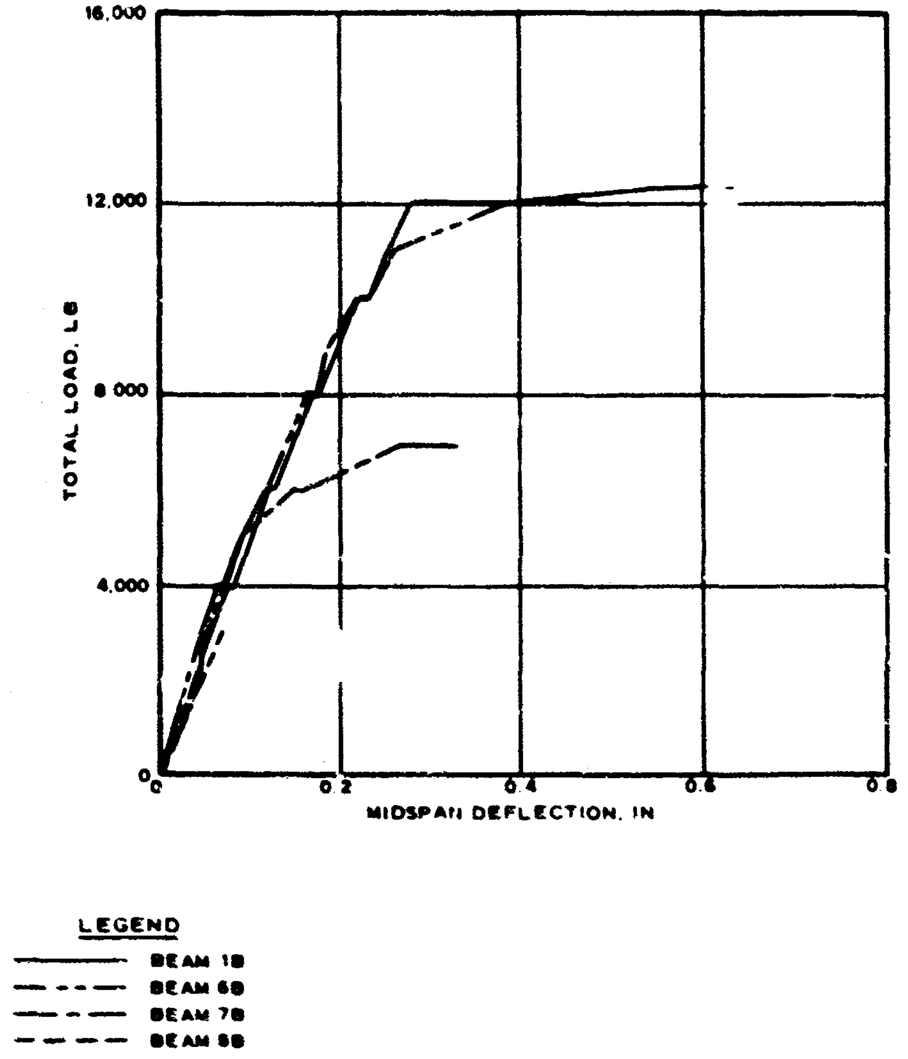


a. EPOXY CONCRETE SERIES



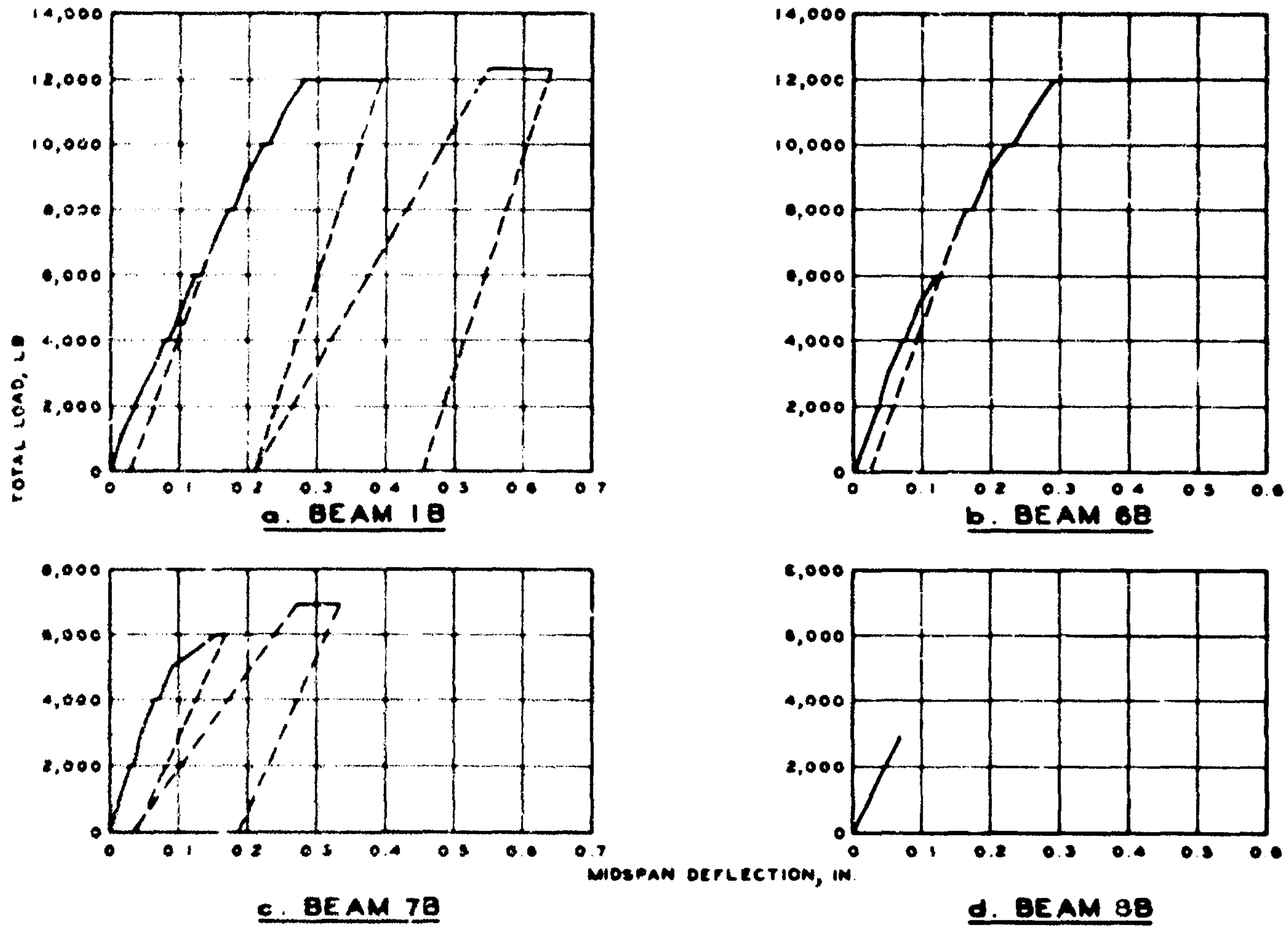
b. POLYESTER CONCRETE SERIES

MOMENT – CURVATURE RELATIONS

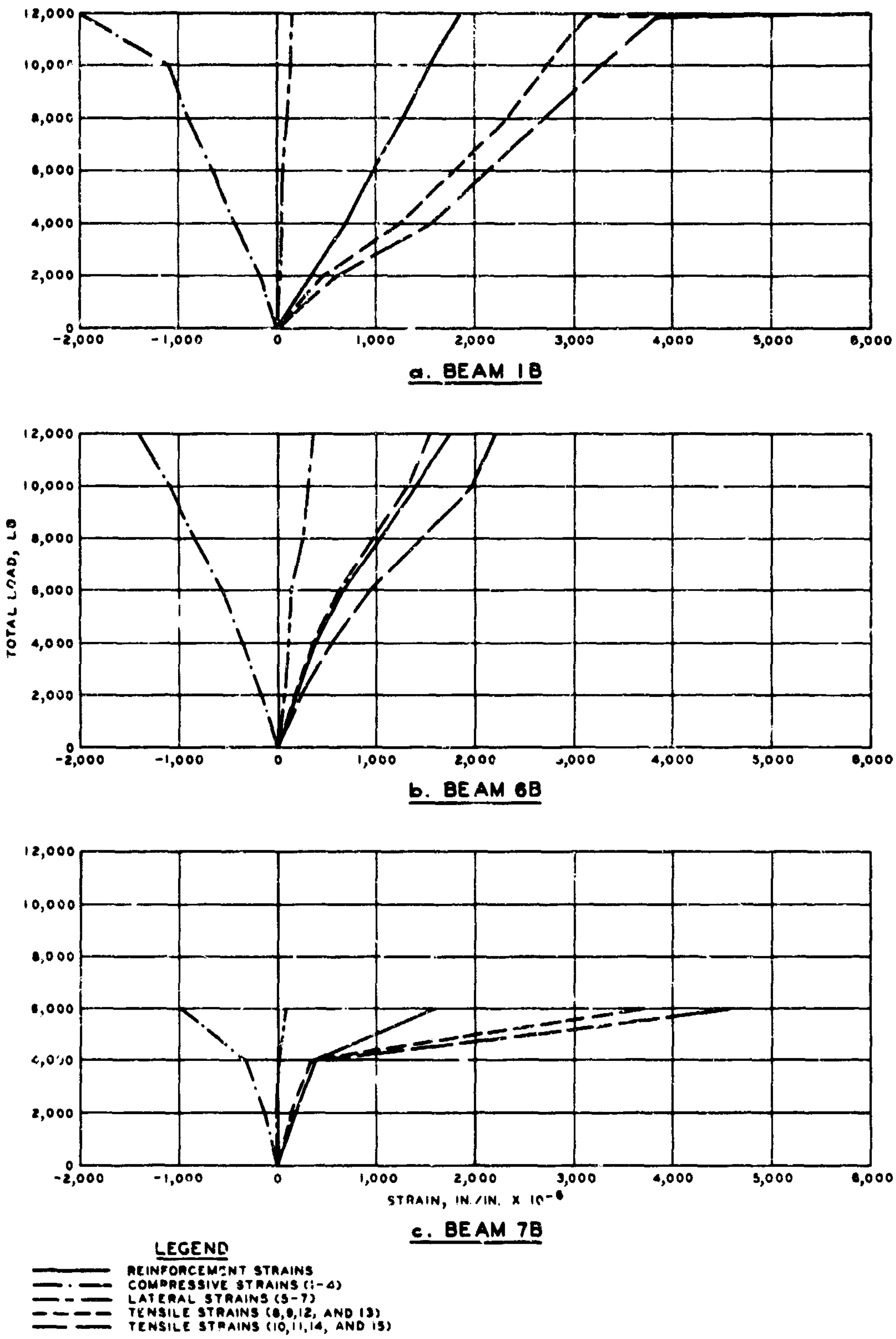


MIDSPAN DEFLECTIONS

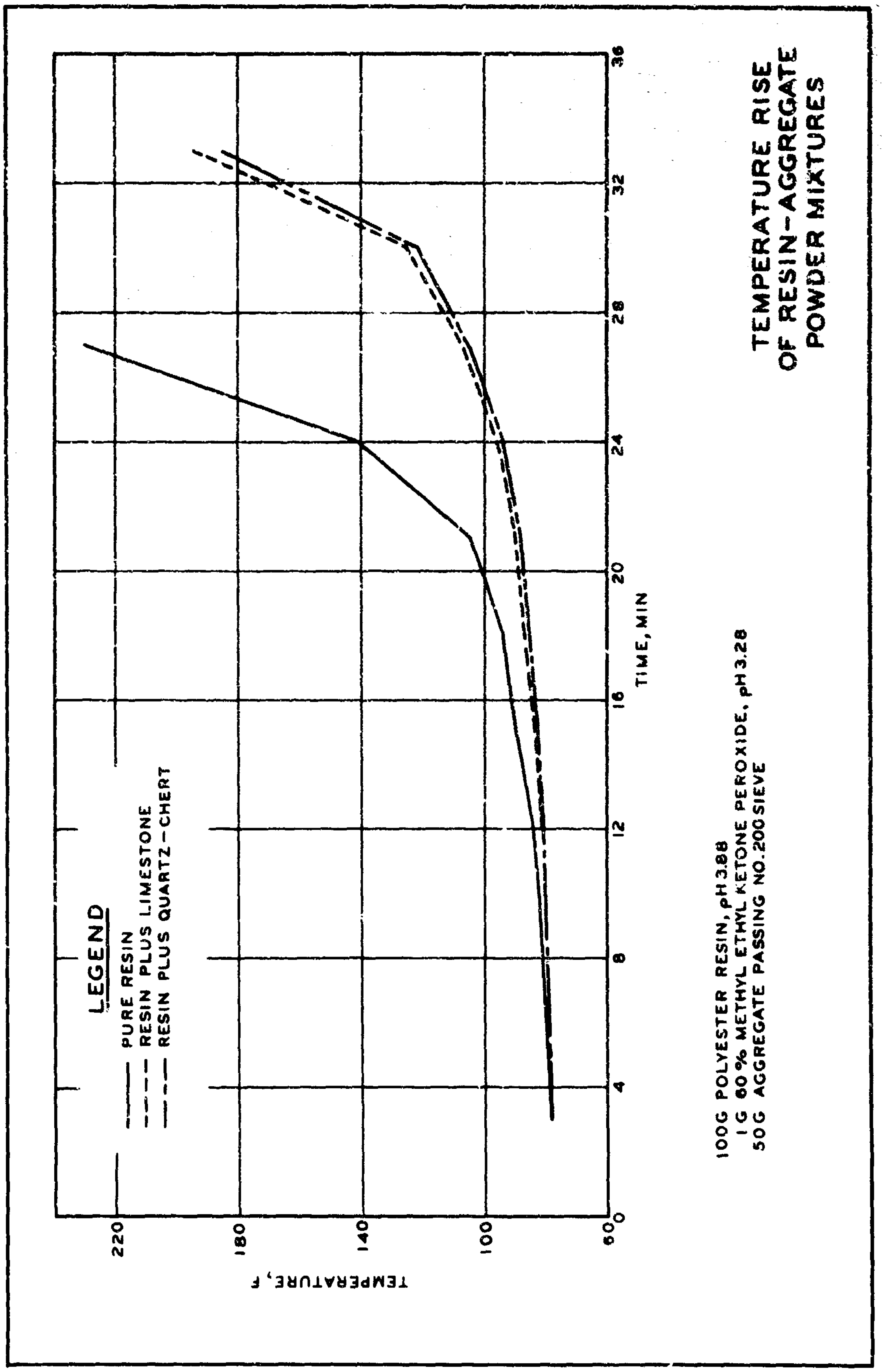
POLYESTER CONCRETE SERIES



LOAD VS MIDSPAN DEFLECTION
POLYESTER CONCRETE SERIES



AVERAGE
STRAIN READINGS
POLYESTER CONCRETE SERIES



TEMPERATURE RISE
OF RESIN-AGGREGATE
POWDER MIXTURES
LEGEND
PURE RESIN
RESIN PLUS LIMESTONE
RESIN PLUS QUARTZ-CHERT
100G POLYESTER RESIN, pH 3.88
1 G 60% METHYL ETHYL KETONE PEROXIDE, pH 3.28
50G AGGREGATE PASSING NO. 200 SIEVE
TIME, MIN
TEMPERATURE, F
0
4
8
12
16
20
24
28
32
36
60
100
140
180
220

PLATE 19

APPENDIX A: ELASTIC ANALYSIS FOR CRACKING OF RESIN CONCRETE

1. For the sake of simplicity, and due to the large variation of resin concrete properties, a linear elastic analysis (similar to the Working Stress Design method in reference 38) was considered satisfactory. This analysis is based on the following assumptions:

   a. Plane sections remain plane (strains are a linear function of the distance from the neutral axis).

   b. All materials are linearly elastic.

   c. Portland cement concrete has zero tensile strength.

   d. A perfect bond exists between the portland cement concrete, the resin concrete, and the reinforcement.

   e. Sections experience pure axial bending.

For these conditions, equation A1 can readily be derived from fig. A1 by fulfilling the plane strain and equilibrium of forces requirements and can be solved for kd .

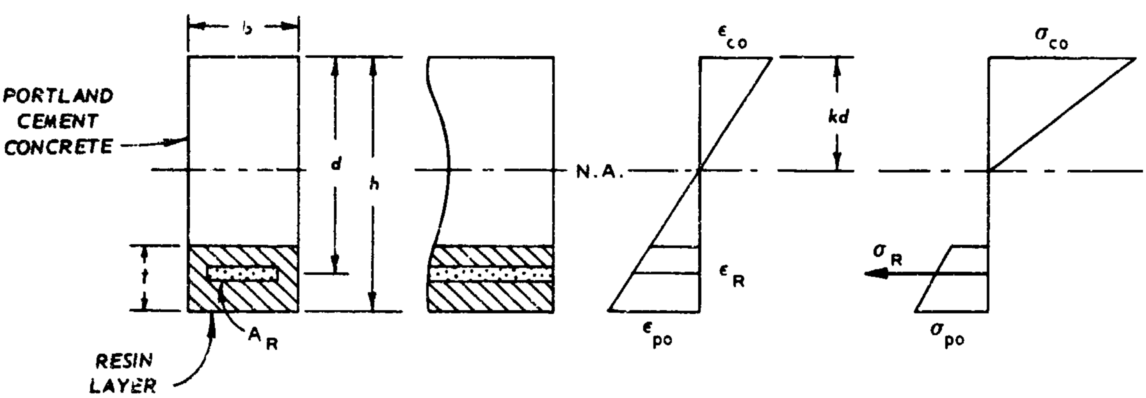


Fig. A1.

$$\epsilon_{co} kd E_c \frac{b}{2} = E_R \frac{\epsilon_{co}}{kd} (d - kd)A_R + E_p \frac{\epsilon_{co}}{kd} \frac{bt}{2} (h - kd + h - kd - t)$$

$$(kd)^2 E_c \frac{b}{2} + kd(E_R A_R + E_p bt) - (E_R A_R d + E_p hbt - E_p \frac{bt^2}{2}) = 0 \qquad \text{(A1)}$$

If ϵ_{pu} is the ultimate resin concrete strain before cracking, we obtain the cracking moment as follows:

$$M_{cp} = \frac{\epsilon_{pu}}{h - kd}(kd)^2\left(E_c\frac{b}{2}\right)\left(\frac{2}{3}kd + d - kd\right) + \frac{\epsilon_{pu}}{h - kd}E_p bt\left[(h - kd - t)\left(h - \frac{t}{2} - d\right) + \left(h - \frac{t}{3} - d\right)\frac{t}{2}\right]$$

$$M_{cp} = \frac{\epsilon_{pu} b}{h - kd}\left\{\frac{(kd)^2}{2}E_c\left(d - \frac{kd}{3}\right) + E_p t\left[(h - kd - t)\left(h - d - \frac{t}{2}\right) + \left(h - \frac{t}{3} - d\right)\frac{t}{2}\right]\right\} \qquad \text{(A2)}$$

where

- A_R = area of tensile reinforcement
- b = width of beam
- d = distance from extreme compression fiber to centroid of tension reinforcement
- E_c = elastic modulus of portland cement concrete
- E_p = elastic modulus of resin concrete (in tension)
- E_R = elastic modulus of reinforcement
- f_y = yield strength
- h = height of beam
- kd = distance from extreme compression fiber to N.A.
- M_{cp} = cracking moment
- M_y = yield moment
- N.A. = neutral axis
- P_{cp} = cracking load (per loading point)
- P_u = ultimate load (per loading point)
- P_y = yield load (per loading point)
- t = thickness of bottom resin concrete layer
- ϵ_{co} = concrete strain in extreme compression fiber
- ϵ_{po} = resin concrete strain in extreme tension fiber
- ϵ_{pu} = ultimate tensile strain in resin concrete at cracking
- ϵ_R = average strain in tensile reinforcement
- σ_{co} = concrete stress in extreme compression fiber

σ_{po} = resin concrete stress in extreme tension fiber

σ_R = average tensile strain in reinforcement

Results of this analysis as well as the calculated yield moments (reference 38, ultimate strength design) are summarized in table A1.

Table A1

Calculated Failure Loads

Beam No.	h in.	d in.	b in.	t in.	A_R $in.^2$	E_c 10^6 psi	E_R 10^6 psi	E_p 10^6 psi	kd in.	ϵ_{pu} 10^{-6}	M_{cp} in.-lb	P_{cp} lb	f_y, psi	M_y in.-lb	P_y lb	Measured P_u, lb
1A	9.00	8.00	3.97	--	0.4	3.70	29.89	--	--	--	--	--	53,500	154,800	$\underline{6450}$	6500
2A	9.00	8.00	3.63	1.35	0.4		29.89	1.23	3.50	1300	135,530	5640	53,500	153,320	$\underline{6390}$	7650
3A	8.91	8.00	3.92	1.42	0.2		29.89		3.00		99,860	$\underline{4160}$	53,500	81,460	3390	4050
4A	9.12	--	3.93	1.56	--		--		2.46		64,700	$\underline{2700}$	--	--	--	3070
5A	9.06	8.00	3.87	1.69	0.36		6.70		2.62		73,040	3040	**NA**	195,070	$\underline{8130}$	8300
6A	8.99	8.00	3.81	2.56	0.4		29.89		3.68		155,706	$\underline{6490}$	53,500	154,160	6420	7250
7A	9.05	8.00	3.98	2.79	0.2		29.89		3.41		110,840	$\underline{4620}$	53,500	81,540	3400	4500
8A	8.99	--	3.72	3.04	--		--		3.00		89,780	$\underline{3740}$	--	--	--	2750
9A	9.02	--	3.98	1.44	--		--		2.38		61,680	$\underline{2570}$	--	--	--	1650
11A	8.99	--	4.00	3.09	--		--		3.02		96,590	$\underline{4020}$	--	--	--	2100
1B	9.10	8.00	3.90	--	0.4	3.12	29.89	--	--	320	--	--	53,500	146,790	$\underline{6120}$	6140
6B	9.00	7.60	3.90	2.88	0.4		29.89	2.07	4.32		49,280	2050	53,500	138,230	$\underline{5760}$	6000
7B	8.90	7.90	3.95	3.00	0.2		29.89		4.09		43,370	1810	53,500	78,500	$\underline{3270}$	3420
8B	9.00	--	3.95	3.00	--		--		3.82		35,405	$\underline{1475}$	--	--	--	1790

Note: **Underscore**- indicate the governing theoretical failure load, i.e. the higher one of the calculated P_{cp} and P_y values. **NA** = not applicable.

Unclassified

Security Classification

DOCUMENT CONTROL DATA - R & D

(Security classification of title, body of abstract and indexing annotation must be entered when the overall report is classified)

1. ORIGINATING ACTIVITY (Corporate author)
U. S. Army Engineer Waterways Experiment Station
Vicksburg, Mississippi

2a. REPORT SECURITY CLASSIFICATION
Unclassified

2b. GROUP

3. REPORT TITLE
USE OF EPOXY OR POLYESTER RESIN CONCRETE IN TENSILE ZONE OF COMPOSITE CONCRETE BEAMS

4. DESCRIPTIVE NOTES (Type of report and inclusive dates)
Final report

5. AUTHOR(S) (First name, middle initial, last name)
Helmut G. Geymayer

6. REPORT DATE
March 1969

7a. TOTAL NO. OF PAGES
84

7b. NO. OF REFS
39

8a. CONTRACT OR GRANT NO.

b. PROJECT NO.

c.

d.

9a. ORIGINATOR'S REPORT NUMBER(S)
Technical Report C-69-4

9b. OTHER REPORT NO(S) (Any other numbers that may be assigned this report)

10. DISTRIBUTION STATEMENT
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11. SUPPLEMENTARY NOTES

12. SPONSORING MILITARY ACTIVITY
Assistant Secretary of the Army (R&D)
Department of the Army

13. ABSTRACT

This report describes the results of an investigation into the feasibility of combining the high compressive strength of portland cement concrete and the superior tensile strength of epoxy or polyester resin concrete into a composite beam. This would increase the beam's flexural strength and improve the corrosion protection for the reinforcement at large deflections by eliminating tensile cracks. The report describes in detail the development of high-strength resin concrete mixtures and summarizes the most important engineering properties of the selected mixtures. Also included are the results of third-point loading tests of 12 reinforced and unreinforced composite beams with 1-1/2- and 3-in.-thick layers of epoxy and polyester resin concretes. These results are compared with results of tests of two reference beams without resin concrete layers and with analytical results. The study led to the following principal conclusions: a. Properly designed resin concrete layers at the tension face of concrete beams can be used to moderately increase the strength and rigidity of reinforced concrete beams, or to upgrade the flexural strength of unreinforced beams by a factor of two to three. b. More important than their influence on strength is the ability of resin concrete layers to provide a noncracking moisture barrier or corrosion protection practically up to beam failure. c. The epoxy resins appeared to be more suitable for this application than the polyester resins investigated due to lower shrinkage and exotherm as well as higher tensile strength and tensile strain capacity. d. In proportioning resin concrete mixtures, early attention should be directed to properties other than strength (such as shrinkage, exotherms, coefficient of thermal expansion, creep, sensitivity to environmental factors, etc.).

DD FORM 1 NOV 65 1473 REPLACES DD FORM 1473, 1 JAN 64, WHICH IS OBSOLETE FOR ARMY USE.

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14. KEY WORDS	LINK A		LINK B		LINK C	
	ROLE	WT	ROLE	WT	ROLE	WT
Beams (supports)						
Concrete beams						
Concretes						
Concrete testing						
Epoxy resins						
High-strength concretes						
Polyester resins						

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